CARIBBEAN UNIFORM BUILDING CODE

PART 1

ADMINISTRATION AND ENFORCEMENT OF THE CODE

Caribbean Community Secretariat
Georgetown
Guyana

1985

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

PREFACE

The need for a building Code related to specific conditions which prevail in the Caribbean has long been recognised. A start is now being made to meet this need with the release of Part 1, Part 2 (except Section 5), and Part 3, of the Caribbean Uniform Building Code (CUBIC). It is planned to publish Parts 4 and 5 of the Code at a later stage. The Code has emerged from the implementation of a major project, involving inter alia, a process of comprehensive and wide ranging consultation among Caribbean experts, acting as consultants and resource persons, and as representatives of governments of Member States of the Caribbean Community, as well as of the full range of non-governmental interests concerned with building standards in the Caribbean.

While relevant concerns and standards of similar Codes in existence elsewhere have been incorporated into the exercise, the Code is uniquely Caribbean in that it speaks directly to the particular requirements of the Caribbean environment, taking special account, for example, of the Region's susceptibility to hurricanes and earthquakes.

Minimum standards for safety of buildings are proposed, and detailed suggestions set out on what should comprise the respective responsibilties of building owners, contractors, and administrative authorities. In order that we in the Caribbean might satisfy these standards and discharge these responsibilities, the Code sets out suggested design criteria, administrative and enforcement requirements relating to materials, and requirements relating to construction practice, occupancy levels, fire safety, and public health.

The Code reflects current knowledge and state of the art. It can thus be anticipated that revisions will periodically be required, as technology changes, and as the results of research efforts become available. It will be a continuing challenge to our commitment to regional cooperation in this field, in the Caribbean, to keep abreast of such developments in future years.

The Code has emerged from concerted collaborative efforts over a number of years among Caribbean experts in the field of building standards, belonging to such organisations as the council of Caribbean Engineering Organisations, the University of the West Indies, the Caribbean Meteorologicl Institute, and the Caribbean Development Bank. Dr. Myron Chin of the University of the West Indies has, as Project Manager, ably directed the exercise to this successful conclusion.

The Project from which this Code is the outcome was funded jointly by the Caribbean Development Bank and the United States Agency for International Development. The Caribbean Community is extremely grateful for this support.

On behalf of the Caribbean Community, I acknowledge the contributions and services of all organisations and individuals whose resources and expertise have combined to give us a Caribbean Uniform Building Code.

Member States of CARICOM now have the benefit of this invaluable tool on which they can draw in formulating or revising their respective national system, as they continue to pursue the goal of improving the standards of environtmental care for the people of the Community.

RODERICK RAINFORD
SECRETARY-GENERAL
CARIBBEAN COMMUNITY SECRETARIAT

April 22, 1986

PART 1

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FOREWORD

The purpose of Building Codes is to set out minimum requirements for safety in buildings. The successful implementation of the Codes depends on the acceptance of and the compliance with the requirements and the establishment of suitable machinery for administration and enforcement. Part I of the Caribbean Uniform Building Code deals with the administration and enforcement of the requirements of the Code.

Most Commonwealth Caribbean countries have administering and enforcing systems for planning laws in force, and it is not anticipated that the provisions of this part of the Code will be in conflict with any of those systems. Included in this part of the Code are the general application of the Code, the limited application situations and prohibitions, and the identification of the responsibilities of building owners, constructors, and administering authorities. The powers of the administering authorities are stated, and the procedures to provide for the smooth processing of applications have been set out.

Unlike other parts of the Code which will be influenced by the advancement of technology, revisions to this part would depend on the need of the Governments to exercise control of the development process. However, the refinement of administration and enforcement should be examined continuously since it is a significant aspect in the implementation of the National Building Code.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

The first number indicates the Part of the Code, the first digit in the second number indicates the Section in the Part, the second and third digits in the second number indicate the Subsection in the Section, and the third number indicates the Article in the Subsection. These are illustrated as follows:

2	- Part 2
2.500	- Part 2, Section 5
2.506	- Part 2, Section 5, Subsection 6
2.506.3	- Part 2, Section 5, Subsection 6, Article 3

PART I

1.100 ADMINISTRATION AND ENFORCEMENT

1.101 Scope

1.101.1 APPLICATION - These requirements apply to the design, construction and occupancy of new buildings, and the alteration, reconstruction, demolition, removal, relocation, maintenance and occupancy of existing buildings except as such matters are otherwise provided for in other written laws or in the rules and regulations for promulgation under the provisions of this Code.

1.101.2 The requirements of the Caribbean Uniform Building Code shall apply to all work falling within the jurisdiction of these regulations.

1.101.3 LIMITED APPLICATION TO EXISTING BUILDINGS - Where a building or any part thereof is altered, these requirements apply to the parts of the building that are altered.

1.101.4 These requirements apply where the whole or any part of a building is relocated either within or into the state.

1.101.5 When the whole or any part of a building is demolished, these requirements apply to the work involved in the demolition and to work required to any parts remaining after demolition to the extent that deficiencies remaining after demolition are corrected.

1.101.6 When a building is damaged by fire, earthquake or other cause, the requirements and the appropriate regulations in the Fire Prevention Bylaw or, in the absence of a Fire Prevention Bylaw, the Building Code apply to the work necessary to reconstruct damaged portions of the building.

1.101.7 When an unsafe condition exists in or about a building, these requirements and the appropriate regulations in the Fire Prevention Bylaw or, in the absence of a Fire Prevention Bylaw, the Building Code apply to the work necessary to correct the unsafe condition.

1.101.8 Where the use of a building or any part thereof is changed, these requirements apply to all parts of the building affected by the change.

1.101.9 EXEMPTIONS - These requirements do not apply to

(a) public works located on a street or on a public transit right of way,

(b) public utility towers and poles, television and radio or other communciation aerials and towers, except for loads resulting from those located on or attached to buildings,

(c) flood control and hydroelectric dams and structures,

(d) mechanical or other equipment and appliances not specifically regulated in these requirements,

(e) accessory buildings not greater than 10 square metres in building area provided they do not create a hazard,

(f) farm buildings of 93 square metres or less other than those used as residences, and

(g) special development projects.

1.101.10 MATTERS NOT PROVIDED FOR - Any requirement essential for structural, fire or sanitary safety of any existing or proposed building or structure, or essential for the safety of its occupants thereof, and of which is not specifically covered by this Code shall be determined by the authority having jurisdiction.

1.102 Prohibitions

1.102.1 Any person who fails to comply with any order or notice issued by an authority having jurisdiction, or who allows a violation of this Code to continue, contravenes the provisions of this Code.

1.102.2 No person shall work or authorize or allow work to proceed on a project for which a permit is required unless a valid permit exists for the work to be done.

1.102.3 No person shall deviate from the approved plans and specifications forming a part of the building permit, or omit or fail to complete, prior to occupancy, work required by the said approved plans and specifications, without first having obtained in writing the approval of the authority having jurisdiction to do so.

1.102.4 No person shall occupy or allow the occupancy of any building, or part thereof, unless the owner has obtained an occupancy certificate from the authority having jurisdiction.

1.102.5 No person having authority in the construction, reconstruction, demolition, alteration, removal, relocation or occupancy of a building shall cause, allow or maintain any unsafe condition.

1.102.6 No person shall excavate or undertake work on, over or under public property, or erect or place any construction or work or store any materials thereon without approval having first been obtained in writing from the appropriate Government Authority.

1.102.7 No person shall allow the property boundaries of a building lot to be so changed as to place a building or part thereof in contravention of this Code.

1.102.8 No person shall knowingly submit false or misleading information.

1.103 Responsibilities of the Owners

1.103.1 Every owner shall allow the authority having jurisdiction to enter any building or premises at any reasonable time for the purpose of administering and enforcing this Code.

1.103.2 Every owner shall obtain all permits or approvals required in connection with proposed work, prior to commencing or continuing the work to which they relate.

1.103.3 Every owner shall ensure that the plans and specifications on which the issuance of the building permit was based are continuously available at the site of the work for inspection during work hours, by the authority having jurisdiction, and the permit, or true copy thereof, is posted conspicuously on the site during the entire execution of the work.

1.103.4 Every owner shall give notice to the authority having jurisdiction of the date on which the owner intends to begin work, not less than two working days prior to commencing work on the building site.

1.103.5 Every owner shall give notice in writing to the authority having jurisdiction, listing

(a) prior to commencing the work, the name, address and telephone number of

(i) the constructor or other person in charge of the work,
(ii) the engineer or architect reviewing the work, and
(iii) any inspection or testing agency engaged to monitor the work, and

(b) any change in or termination of employment of such persons during the course of the construction immediately that such change or termination occurs.

1.103.6 Every owner shall give notice to the authority having jurisdiction

(a) of intent to do work that has been ordered inspected during construction,

(b) of intent to cover work that has been ordered to be inspected prior to covering, and

(c) when work has been completed so that a final inspection can be made.

1.103.7 Every owner shall give notice in writing to the authority having jurisdiction

(a) immediately that any change in ownership or change in the address of the owner occurs prior to the issuance of an occupancy permit, and

(b) prior to occupying any portion of the building if it is to be occupied in stages.

1.103.8 Every owner shall give such other notice to the authority having jurisdiction as may be required by the provisions of this Code.

1.103.9 Every owner shall make or have made at the owner's expense tests or inspection as necessary to prove compliance with this Code and shall promptly file a copy of all such tests or inspection reports with the authority having jurisdiction.

1.103.10 Every owner shall provide an up-to-date survey of the building site when and as required by the authority having jurisdiction.

1.103.11 When required by the authority having jurisdiction, every owner shall uncover and replace at the owner's expense any work that has been covered contrary to an order issued by the authority having jurisdiction.

1.103.12 Every owner is responsible for the cost of repair of any damage to public property or work located thereon that may occur as a result of undertaking work for which a permit is required.

1.103.13 No owner shall deviate from this Code or from the conditions of the permit to omit work required by this Code or the conditions of the permit without first obtaining from the authority having jurisdiction permission in writing to do so.

1.103.14 Every owner shall obtain an occupancy certificate from the authority having jurisdiction prior to any

(a) occupancy of a building or part thereof after construction, partial demolition or alteration of the building, or

(b) change in the occupancy of any building or part thereof.

1.103.15 Every owner shall ensure that no unsafe condition exists or will exist because of the work being undertaken or not completed should occupancy occur prior to the completion of any work being undertaken that requires a permit.

1.103.16 When required by the authority having jurisdiction, every owner shall provide a letter to certify compliance with this Code and of any permits required.

1.103.17 The granting of a permit, the approval of the plans and specifications or inspections carried out by the authority having jurisdiction shall not in any way relieve the owner of a building from full responsibility for carrying out the work or having the work carried out in accordance with this Code, including ensuring that the occupancy of the building, or any part thereof, is in accordance with the terms of the occupancy certificate, and including compliance with any special conditions made under the provisions of 1.106.6.

1.103.18 When a building or part thereof is in an unsafe condition, the owner shall forthwith take all necessary action to put the building in a safe condition.

1.104 Responsibilities of the Constructor

1.104.1 Every constructor shall ensure that all construction safety requirements of this Code are complied with.

1.104.2 Every constructor is responsible for ensuring that no excavation or other work is undertaken on public property, and that no building is erected or materials stored in whole or in part thereon without approval first having been obtained in writing from the appropriate government authority.

1.104.3 Every constructor is responsible jointly and severally with the owners for any work actually undertaken.

1.105 Responsibilities of the Authority having Jurisdiction

1.105.1 The authority having jurisdiction is responsible for the administration and enforcement of this Code.

1.105.2 The authority having jurisdiction shall keep copies of all applications received, permits and orders issued, inspections and tests made and of all papers and documents connected with the administration of this Code for such time as other regulations require.

1.105.3 The authority having jurisdiction shall accept any construction or condition that lawfully existed within the area of jurisdiction prior to the effective date of this Code provided that such construction or condition does not constitute an unsafe condition.

1.105.4 The authority having jurisdiction shall issue in writing such notices or orders as may be necessary to inform the owner where a contravention of this Code has been observed.

1.105.5 The authority having jurisdiction shall answer such relevant questions as may be reasonable with respect to the provisions of this Code when requested to do so but, except for standard design aids, shall refrain from assisting in the laying out of any work and from acting in the capacity of a consultant.

1.105.6 The authority having jurisdiction shall issue a permit to the owner when, to the best of his knowledge, the applicable conditions as set forth in this Code have been met.

1.106 Powers of the Authority having Jurisdiction

1.106.1 The authority having jurisdiction may enter any building or premises at any reasonable time for the purpose of administering or enforcing this Code.

1.106.2 The authority having jurisdiction is empowered to order

(a) a person who contravenes this Code to comply with it within the period specified,

(b) work to stop on the building or any part thereof if such work is proceeding in contravention of this Code, or if there is deemed to be an unsafe condition,

(c) the removal of any unauthorized encroachment on public property,

(d) the removal of any building or part thereof constructed in contravention of this Code,

(e) the cessation of any occupancy in contravention of this Code,

(f) the cessation of any occupancy if any unsafe condition exists because of work being undertaken or not completed, and

(g) the correction of any unsafe condition.

1.106.3 The authority having jurisdiction may direct that tests of materials, equipment, devices, construction methods, structural assemblies or foundation conditions be made, or sufficient evidence or proof be submitted at the expense of the owner, where such evidence or proof is necessary to determine whether the material, equipment, device, construction or foundation condition satisfies this Code.

1.106.4 The authority having jurisdiction may require any owner to submit an up-to-date plan or survey prepared by a registered land surveyor which shall contain sufficient information regarding the site and the location of any building

(a) to establish before construction begins that all the requirements of this Code in relation to this information will be complied with, and

(b) to verify upon completion of the work that all such requirements of this Code have been complied with.

1.106.5 When site conditions, size or complexity of the work warrant, the authority having jurisdiction may require that all plans and specifications or any part therof be certified, and the construction or alteration of any building or part thereof be generally reviewed during construction, by a professionally qualified architect or engineer.

1.106.6 The authority having jurisdiction may, subject to conditions, issue a permit when in his opinion it is warranted with respect to a building or occupancy in which existing construction is not in complete compliance with this Code.

1.106.7 The authority having jurisdiction may issue a permit at the risk of the owner, with conditions if necessary to ensure compliance with this Code and any other applicable regulations, to excavate or to construct a portion of a building before the entire plans of the whole project have been submitted or accepted.

1.106.8 The authority having jurisdiction may issue a permit for the entire project conditional upon the submission, prior to commencing work thereon, of additional information not available at the time of issuance, if such data are of secondary importance and are of such nature that the withholding of the permit until its availability would unreasonably delay the work.

1.106.9 The authority having jurisdiction may refuse to issue any permit

(a) whenever information submitted is inadequate to determine compliance with the provisions of this Code,

(b) whenever incorrect information is submitted,

(c) that would authorize any building work or occupancy that would not be permitted by this Code, or

(d) that would be prohibited by any other regulation.

1.106.10 The authority having jurisdiction may revoke a permit if

(a) there is a contravention of any condition under which the permit was issued,

(b) the permit was issued in error, or

(c) the permit was issued on the basis of incorrect information.

1.106.11 The authority having jurisdiction may issue a temporary occupancy certificate, subject to compliance with provisions to safeguard persons in or about the premises, to allow the occupancy of a building or a part thereof for the accepted use, prior to commencement or completion of the construction or demolition work.

1.106.12 When any building, construction or excavation or part thereof is in an unsafe condition as a result of being open or unguarded, or because of danger from fire or risk of accident because of its ruinous or dilapidated state, faulty construction, abandonment or otherwise, and when due notice to correct such condition has not been complied with, the authority having jurisdiction may

(a) demolish, remove or make safe such building, construction, excavation or part thereof at the expense of the owner and may recover such expense in like manner as municipal taxes, and

(b) take such other measures as it may consider necessary to protect the public.

1.106.13 When immediate measures must be taken to avoid an imminent danger of fire or risk of accident, the authority having jurisdiction may take such action as is appropriate, without notice, and at the expense of the owner.

1.106.14 Before issuing an occupancy permit, the authority having jurisdiction may require the owner to provide letters to certify that the requirements of this and the necessary permits have been met.

1.106.15 The authority having jurisdiction shall issue the owner an order or notice in writing to correct any unsafe condition observed in any building.

1.106.16 The authority having jurisdiction shall provide in writing, when requested to do so, all reasons for refusal to grant a permit.

1.106.17 Where any failure occurs which causes or has the potential to cause injury or loss of life, the authority having jurisdiction may require the owner or occupier to submit a report stating

(a) the name and address of the owner of the building,

(b) the address or location of the building involved in the failure,

(c) the name and address of the constructor, and

(d) the nature of the failure.

1.107 Permits and Permit Fees

1.107.1 PERMITS - A permit is required whenever work regulated by this Code is to be undertaken.

1.107.2 Permits, in addition to the permits required in 1.107.1 with respect to building components and services, may be required by the authority having jurisdiction.

1.107.3 An occupancy certificate is required

(a) to allow the occupancy of a building or part thereof, or

(b) when the occupancy of a building or part thereof is changed.

1.107.4 APPLICATION FOR A PERMIT - To obtain a permit the owner shall file an application in writing on a prescribed form.

1.107.5 Except as otherwise allowed by the authority having jurisdiction, every application shall

(a) identify and describe in detail the work and occupancy to be covered by the permit for which application is made,

(b) describe the land on which the work is to be done by a description that will readily identify and locate the building lot,

(c) include plans and specifications, unless otherwise accepted by the authority having jurisdiction, and show the occupancy of all parts of the building,

(d) be accompanied by the required fee, and

(e) state the names, addresses and telephone numbers of the owner, architect, engineer or other designer and constructor.

1.107.6 An application for a permit may be deemed to have been abandoned 6 months after the date of filing, unless such application is being actively pursued by the applicant.

1.107.7 APPROVAL IN PART - Where in order to expedite work, approval of a portion of the building is desired prior to the issuance of a permit for the whole project, application shall be made for the complete project, and detailed plans and specifications covering the portion of the work for which immediate approval is desired shall be filed with the authority having jurisdiction.

1.107.8 Should a permit be issued for part of a building, the holder of such permit may proceed without assurance that the permit for the entire building will be granted.

1.107.9 After issuance of the permit, application may be made for revision of the permit, and such application shall be made in the same manner as for the original permit.

1.107.10 A permit shall expire and the right of an owner under the permit shall terminate if

(a) the work authorized by the permit is not commenced within 24 months from the date of issuance of the permit and actively carried out thereafter, or

(b) work is suspended for a period of 24 months.

1.107.11 Permit fees and all procedures and conditions applicable thereto shall be as established by the authority having jurisdiction.

1.107.12 PERMIT FOR A TEMPORARY BUILDING - Notwithstanding anything contained elsewhere in this Code, a permit for a temporary building may be issued by the authority having jurisdiction, authorizing for a limited time only the erection and existence of a building or part thereof for an occupancy which because of its nature will exist for a short time under circumstances which warrant only selective compliance with the requirements.

1.107.13 A permit for a temporary building shall state the date after which and the conditions under which the permit is no longer valid.

1.107.14 A permit for a temporary building may be extended provided permission in writing is granted by the authority having jurisdiction.

1.107.15 A permit for a temporary building shall be posted on the building.

1.107.16 TENTS AND AIR-SUPPORTED STRUCTURES - Except where permitted by the authority having jurisdiction, a tent or air-supported structure shall not be erected unless a certificate of occupancy has been obtained.

1.107.17 The authority having jurisdiction shall issue a certificate of occupancy when it is satisfied that a tent or air-supported structure complies with the Caribbean Uniform Building Code.

1.107.18 The certificate of occupancy for a tent or air-supported structure is renewable every 12 months provided the tent or air-supported structure complies with all laws and regulations in effect at the time of request for renewal.

1.108 Inspection

1.108.1 The designer shall make copies of all inspection and review reports available to the authority having jurisdiction upon request.

1.108.2 The person responsible for foundation review shall prepare and sign a report of each review and send the report as soon as practical to the authority having jurisdiction.

1.108.3 The authority having jurisdiction shall be notified and given an opportunity to inspect the site before excavation, before a foundation is placed and before a superstructure is placed on a foundation.

1.109 Environmental Factors

1.109.1 The environmental factors which are to be taken into consideration in the design of buildings are stated in Part 2 of the Code - "Structural Design Requirements".

1.110 Board of Appeal

1.110.1 Unless legislation provides for a Board of Appeal, the authority having jurisdiction may create such a board, and if created the Board shall have the following establishment and terms of reference.

1.110.2 The Board of Appeal shall consist of at least 7 members, including a Chairman and Vice-Chairman, who are qualified by experience and training to consider matters pertaining to building design and construction.

1.110.3 The members of the Board of Appeal shall be appointed for such periods as may be deemed appropriate.

1.110.4 In the absence of the Chairman and Vice-Chairman at any meeting, the members shall elect a Chairman from among those present to preside.

1.110.5 A Secretary without voting privileges shall be appointed and be responsible for maintaining records, correspondence and keeping minutes of the Board meetings.

1.110.6 Three members of the Board constitute a quorum.

1.110.7 The Board shall hear appeals from decisions of the designated official pertaining to this Code and shall confirm, alter or reject such decisions in accordance with the intent of this Code.

1.110.8 An appeal against a decision of the designated official may be submitted to the Board of Appeal by a person who

(a) has applied under the provisions of this Code for a permit which has not been granted,

(b) has had a permit revoked, or

(c) feels himself adversely affected by a decision of the designated official.

1.110.9 The appellant shall file with the Secretary of the Board of Appeal a statement in writing in such detail as will enable the Board properly to consider the appeal, addressed to the Board of Appeal and setting out

(a) the nature and subject matter of the appeal,

(b) the address of the building affected by the appeal, and

(c) the sections of these requirements applicable to the appeal.

1.110.10 The Board shall meet as often as it deems necessary to conduct work of the Board in an expeditious manner.

1.110.11 The Board shall consider any appeal filed with the Secretary within 15 calendar days after a decision of the designated official and need not consider any appeal filed thereafter.

1.110.12 The Board shall meet to hear an appeal within 45 calendar days after the appeal has been filed, or such lesser time as may be designated in the notice creating the Board.

1.110.13 The Board shall communicate its decision in writing to the person making the appeal within 5 working days after the decision has been reached and shall form part of the public records.

1.110.14 The appellant and the designated official or their respective representatives may attend the meeting of the Board and may make representations concerning the matter under appeal.

1.110.15 The Board in making its decisions may inform itself in any manner it deems appropriate and which it considers necessary to arrive at a decision.

1.110.16 The Board shall inform the appellant and designated official of any additional information considered and shall provide an opportunity for either party to respond.

1.110.17 The decisions of a majority of the members of the Board present at a hearing shall be the decision of the Board, whose decision shall not be subject to further appeal.

1.110.18 A person who appeals in accordance with the provisions of this section shall not be relieved of complying with the requirements pending a decision on the appeal, nor shall the authority having jurisdiction be precluded from enforcing the provisions of the requirements during such period.

1.111 Effective Date

1.111.1 These requirements shall come into effect when proclaimed by the authority having jurisdiction.

1.111.2 Any person who carries out work under terms of these requirements after its adoption but before its effective date may do so, but all such work shall comply entirely with this Code.

1.111.3 The requirements of this Code or any amendments shall not affect the rights of an owner under a permit issued prior to the effective date of this Code or such amendment provided the owner has commenced work within 24 months of the date of issuance of the permit and has actively carried out work thereafter.

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 1

Structural Design Requirements
DEAD LOAD AND GRAVITY LIVE LOAD

1985

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 1
DEAD LOAD AND GRAVITY LIVE LOAD

Caribbean Community Secretariat
Georgetown
Guyana

1985

PART 2

SECTION I

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TABLES

FOREWORD

The unit weights of basic materials vary from territory to territory in the Commonwealth Caribbean and therefore when they are used in calculating dead loads they must be properly substantiated. Ranges of unit weights for aggregate timber and concrete are provided in Part 2 Sections 6 and 8 of this Code and these should be carefully noted.

In cases where unit weights of materials cannot be adequately substantiated this section stipulates that the latest edition of British Standard 648, "Schedule of Weights of building Materials" or other equivalent authoritative standard be used.

The appropriate loads for different uses of parts of a building or structure are fully detailed in this section. All conceivable uses have been listed but recommendations have been made for consideration to be given in the design where crowded conditions are likely to occur such as in living rooms during Caribbean-style parties, and structures adjacent to sports grounds where there is access to the roof.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

The first number indicates the Part of the Code, the first digit in the second number indicates the Section in the Part, the second and third digits in the second number indicate the sub-section in the Section, and the third number indicates the Article in the sub-section. These are illustrated as follows:

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ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

PART 2

SECTION 1

2.100 DEAD LOAD AND GRAVITY LIVE LOAD

2.101 Scope

2.101.1 This section of the Caribbean Uniform Code gives recommended minimum dead and gravity live loads for the design of buildings.

2.101.2 Special cases of loading due to vibration, moving equipment or temporary construction loading are not dealt with in this section.

2.101.3 This section does not include recommendations for wind load, and earthquake loads, which are dealt with in Part 2, Sections 2 and 3.

2.102 Definitions

2.102.1 For the purpose of this section the following definitions shall apply:

Dead Load means the weight of all walls, partitions, floors, roofs and finishes including all other permanent construction.

Gravity Live Load means all loads other than dead loads, wind loads and earthquake loads.

Mass means the quantity of matter contained in a body.

The Newton is the SI unit of force. The weight of a body of mass m kg is m x 9.80 Newtons, taking the acceleration due to gravity as 9.80 m/s^2.

Weight means the force due to gravity acting on or through any part of a building.

2.103.1 Dead Loads

2.103.1 The unit weights of basic materials used in the calculation of dead loads shall preferably be based on properly substantiated information. Where this is not available, the values given in the latest edition of British Standard 648, "Schedule of Weights of Building Materials", or an equivalent authoritative standard shall be used.

2.103.2 In the case of aggregate, concrete and timber, the unit weights of which are known to vary between different territories in the Caribbean, special note shall be taken of the range of unit weight of these building materials as given in Part 2 Sections 6 and 8.

2.103.3 The unit weight of reinforced concrete will vary from that of plain concrete depending on percentage of reinforcing steel.

2.103.4 Where partitions are shown on the plans, their actual weights shall be included in the dead load. To provided partitions where their positions are not shown on the plans, the beams and floor slabs where these are capable of effective lateral distribution of the load, shall be designed to carry in addition to other loads, distributed load per square metre not less than one-third of the weight per metre run of the finished partitions and for offices not less than 1 kN/m^2.

2.103.5 The weight of tanks and other receptacles, and their contents, shall be considered as dead load; account shall be taken of the load conditions when a tank or receptacle is full and when it is empty.

2.104 Imposed Floor Loads

2.104.1 Table 2.104.1 gives the loads appropriate for the different uses to which the parts of a building or structure may be put. The distributed loads are the equivalent uniformly distributed static loads per square metre of plan area and provide for the normal effects of impact and acceleration, but not for any special concentrated loads.

2.104.2 All floor slabs shall be designed to carry the appropriate distributed or concentrated imposed load as given in Table 2.104.1 whichever produces the greater stresses given in the part of the floor slab under consideration.

2.104.3 In the design of floor slabs, concentrated loads shall be considered to be applied in the positions which produce the maximum stresses, and where deflection is the design criterion, in the positions which produce maximum deflections. The concentrated imposed load need not be considered where the floor slabs are capable of effective lateral distribution of this load.

Note 1 - Consideration should be given in the design of living rooms where crowded conditions are likely to occur during Caribbean-type fetes or parties.

Note 2 - Consideration should be given in the design of structures adjacent to sports grounds for crowd loads where there is a possible access to the roof.

Note 3 - The designer should consider higher access corridor loading than specified in Table 2.104.1 where deemed necessary.

TABLE 2.104.1
USES AND LOADS

Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
	kN/m^2	kN
1-1 ART GALLERY (See Museum Floors)		
1-2 ASSEMBLY BUILDINGS such as public halls and theatres, but excluding drill halls, places of worship, public lounges, schools and toilet room:		
with fixed seating	4.0	
without fixed seating*	5.0	3.6
1-3 BALCONIES (See also Residential Buildings)	Same as the rooms to which they give access	1.5 per metre run concentrated at the edge
1-4 BANKING HALLS	3.0	-
1-5 BEDROOMS:		
Domestic buildings (see also Residential Buildings)	1.5	1.4
Hotels and motels	2.0	1.8
Institutional buildings	1.5	1.8
1-6 BILLIARD ROOMS	2.0	2.7
1-7 BOILER ROOMS	7.5	4.5
1-8 BOOKSTORES	2.4 for each metre of storage height	7.0
1-9 BROADCASTING STUDIOS:		
Corridors (See Corridors)		
Dressing rooms	2.0	1.8
Fly galleries	4.5 kN per metre run uniformly distributed over the width	-
Grids	2.5	-
Stages	7.5	4.5
Studios	4.0	-
Toilet rooms	2.0	-
1-10 BUNGALOWS	1.5	1.4
1-11 CATWALKS	Concentrated loads only	1.0 at 1.0 m centres
1-12 CHAPELS AND CHURCHES	3.0	2.7
1-13 CINEMAS (See Assembly Buildings and Broadcasting Studios)		

* Fixed seating implies that the removal of the seating and the use of the space for other purposes is improbable.

NOTE - Load/m^2 = force/m^2 unit of which is pascal
1 pascal (Pa) = 1 N/m^2

TABLE 2.104.1 (Continued)

	Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
		kN/m^2	kN
1-14	CLASS-ROOMS	3.0	2.7
	Where class-rooms may be used as places of assembly the values shown under Colleges shall be used		
1-15	CLUBS		
	Assembly areas with fixed seating*	4.0	-
	Assembly areas without fixed seating	5.0	3.6
	Bedrooms	1.5	1.8
	Billiard Rooms	2.0	2.7
	Corridors (See Corridors)		
	Dining rooms	2.0	2.7
	Kitchens	to be determined, but not less than 3.0	4.5
	Lounges	2.0	2.7
	Laundries	3.0	4.5
	Toilet rooms	2.0	-
1-16	COLD STORAGE	5.0 for each metre of storage height with a minimum of 15.0	9.0
1-17	COLLEGES:		
	Assembly areas with fixed seating*	4.0	-
	Assembly areas without fixed seating*	5.0	3.6
	Bedrooms	1.5	1.8
	Class-rooms	3.0	2.7
	Corridors (See Corridors)		
	Dining rooms	2.0	2.7
	Dormitories	1.5	1.8
	Gymnasia	5.0	3.6
	Kitchens	to be determined, but not less than 3.0	4.5
	Laboratories including equipment	to be determined, but not less than 3.0	4.5
	Stages	5.0	3.6
	Toilet rooms	2.0	-

* Fixed seating implies that the removal of the seating and the use of the space for other purposes is improbable.

TABLE 2.104.1 (Continued)

Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
	kN/m^2	kN
1-18 CORRIDORS, HALLWAYS, PASSAGEWAYS, AISLES, PUBLIC SPACES AND FOOTBRIDGES BETWEEN BUILDINGS:		
Buildings subject to crowd loading, except grandstands;	4.0	4.5
Buildings subject to loads greater than from crowds including wheeled vehicles trolleys and the like	to be determined, but not less than 5.0	to be determined, but not less than 4.5
All other buildings	Same as the rooms to which they give access	Same as the rooms to which they give access
1-19 DANCE HALLS	5.0	3.6
1-20 DEPARTMENTAL STORES Ship floors for the display and sale of merchandise	4.0	3.6
1-21 DORMITORIES	1.5	1.8
1-22 DRILL ROOMS AND DRILL HALLS	5.0	to be determined, but not less than 9.0
1-23 DRIVEWAYS AND VEHICLE RAMPS Other than in garages for the parking only of passenger vehicles and light vans not exceeding 2 1/2 tonnes (2,500 kg) gross weight	to be determined, but not less than 5.0	to be determined, but not less than 9.0
1-24 DWELLINGS	1.5	1.4
1-25 FACTORIES AND SIMILAR BUILDINGS	5.0 7.5 or 10.0 as appropriate	4.5 6.7 9.0
1-26 FILE ROOMS IN OFFICES	to be determined, but not less than 5.0	to be determined
1-27 FLATS	1.5	1.4
1-28 FOOTPATHS, TERRACES AND PLAZAS leading from ground level:		
No obstruction to vehicular traffic	5.0	9.0
Used only for pedestrian traffic	4.0	4.5

TABLE 2.104.1 (Continued)

	Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
		kN/m^2	kN
1-29	FOUNDRIES	to be determined, but not less than 20.0	-
1.30	GARAGES:		
	Car parking only, for passenger vehicles and light vans not exceeding 2 1/2 tonnes (2,500 kg) gross weight, including driveways and ramps	2.5	9.0
	All repair workshops for all types of vehicles and parking for vehicles exceeding 2 1/2 tonnes, (2,500 kg) gross weight, including driveways and ramps	5.0	-
1-31	GRANDSTANDS:		
	Assembly areas with fixed seating*	4.0	-
	Assembly areas without fixed seating	5.0	3.6
	Corridors and passageways	5.0	4.5
	Toilet rooms	2.0	-
1-32	GYMNASIA	5.0	3.6
1-33	HALLS:		
	Corridors, Hallways and Passageways (See Corridors)		
	Dressing rooms	2.0	1.8
	Fly galleries	4.5 kN per metre run uniformly distributed over the width	-
	Grids	2.5	-
	Projection rooms	5.0	-
	Stages	5.0	3.6
	Toilet rooms	2.0	-
1-34	HOSPITALS:		
	Bedrooms and wards	2.0	1.8
	Corridors, Hallways and Passageways (See Corridors)		
	Dining rooms	2.0	2.7
	Kitchens	to be determined, but not less than 3.0	4.5

* Fixed seating implies that the removal of the seating and the use of the space for other purposes is improbable.

TABLE 2.104.1 (Continued)

	Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
		kN/m^2	kN
	HOSPITALS (continued)		
	Laundries	3.0	4.5
	Toilet rooms	2.0	-
	Utility rooms	2.0	4.5
	X-ray rooms and operating theatres	2.0	4.5
1-35	HOTELS AND MOTELS:		
	Bars and vestibules	5.0	-
	Bedrooms	2.0	1.8
	Corridors, Hallways and Passageways (See Corridors)		
	Dining rooms	2.0	2.7
	Kitchens	to be determined, but not less than 3.0	4.5
	Laundries	3.0	4.5
	Lounges	2.0	2.7
	Toilet rooms	2.0	-
1-36	HOUSES	1.5	1.4
1-37	INDOOR SPORTING FACILITIES:		
	Areas for equipment	to be determined, but not less than 2.0	
	Assembly areas with fixed seating*	4.0	-
	Assembly areas without fixed seating	5.0	3.6
	Corridors (See Corridors)		
	Dressing rooms	2.0	1.8
	Gymnasia	5.0	3.6
	Toilet rooms	2.0	-
1-38	INSTITUTIONALS BUILDINGS		
	Bedrooms	1.5	1.8
	Communal Kitchens	to be determined, but not less than 3.0	4.5
	Corridors, Hallways and Passageways (See Corridors)		
	Dining rooms	2.0	2.7
	Dormitories	1.5	1.8

* Fixed seating implies that the removal of the seating and the use of the space for other purposes is improbable.

TABLE 2.104.1 (Continued)

	Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
		kN/m^2	kN
	INSTITUTIONAL BUILDINGS (continued)		
	Laundries	3.0	4.5
	Lounges	2.0	2.7
	Toilet rooms	2.0	-
1-39	KITCHENS		
	Other than in domestic buildings including normal equipment	to be determined, but not less than 3.0	4.5
1-40	LABORATORIES including equipment	3.0	4.5
1-41	LANDINGS*	Same as the floor to which they give access	Same as the floor to which they give access
1-42	LAUNDRIES		
	Other than in domestic buildings excluding equipment	to be determined, but not less than 3.0	4.5
1-43	LIBRARIES:		
	Reading rooms without book storage	2.5	4.5
	Rooms with book storage (e.g. public lending libraries)	4.0	4.5
	Stack rooms	2.4 for each metre of stack height with a minimum of 6.5	7.0
	Dense mobile stacking on mobile trucks	4.8 for each metre of stack height with a minimum of 9.6	7.0
			-
	Corridors	4.0	4.5
	Toilet rooms	2.0	-
1-44	MACHINERY HALLS		
	Circulation spaces therein	4.0	4.5
1-45	MAISONETTES	1.5	1.4
1-46	MOSQUES (See places of worship)		

TABLE 2.104.1 (Continued)

Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
	kN/m^2	kN
1-47 MOTOR ROOMS, FAN ROOMS and the like, including weight of machinery	to be determined, but not less than 7.5	-
1-48 MUSEUM FLOORS AND ART GALLERIES for exhibition purposes	to be determined, but not less than 4.0	4.5
1-49 OFFICES:		
Corridors and public spaces (See Corridors)		
Filing and storage spaces	5.0	4.5
Offices for general use	2.5	2.7
Offices with computing, data processing and similar equipment	3.5	4.5
Toilet rooms	2.0	-
1-50 PAVEMENT LIGHTS	to be determined, but not less than 5.0	1 1/2 times the wheel load but not less than 9.0
1-51 PLACES OF WORSHIP		
Without fixed seating*	4.0	-
with fixed seating	3.0	2.7
1-52 PRINTING PLANTS:		
Paper storage	to be determined,but not less than 4.0 for each metre of storage height	9.0
Type storage and other areas	to be determined, but not less than 12.5	9.0
1-53 PUBLIC HALLS (See Halls)		
1-54 PUBLIC LOUNGES	2.0	2.7
1-55 RESIDENTIAL BUILDINGS Such as apartment houses, boarding houses, guest houses, hostels, lodging houses and residential clubs, but excluding hotels and motels:		
Bedrooms	1.5	1.8
Communal kitchens	to be determined, but not less than 3.0	4.5

* Fixed seating implies that the removal of the seating and the use of the space for other purposes is improbable.

TABLE 2.104.1 (Continued)

	Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
		kN/m^2	kN
	RESIDENTIAL BUILDINGS (continued)		
	Corridors, hallways and Passageways (See Corridors)		
	Dining rooms and public rooms	2.0	2.7
	Dormitories	1.5	1.8
	Laundries	3.0	4.5
	Toilet rooms	2.0	-
	In all spaces normally used for entertainment	4.0	-
1-56	SCHOOLS (See Colleges)		
1-57	SHOP FLOORS		
	For the display and sale of merchandise	4.0	3.6
1-58	STAIRS:		
	Dwellings not over 3-storey	1.5	1.8
	All other buildings	same as the floors to which they give access but not less than 3.0 and not more than 5.0	same as the floors to which they give access
1-59	STATIONERY STORES	4.0 for each metre of storage height	9.0
1-60	STORAGE (Other than types listed separately under book stores, cold storage, departmental stores, file rooms, stock rooms)	to be determined but not less than 2.4 for each metre of storage height	7.0
1-61	TELEVISION STUDIOS (See Broadcasting Studios)		
1-62	TEMPLE (See Places of Worship)		
1-63	THEATRES (See Assembly Building and Broadcasting Studios)		

TABLE 2.104.1 (Continued)

Use to which building or structure is to be put	Intensity of distributed load	Concentrated load to be applied over any square with a 300 mm side, unless otherwise stated
	kN/m^2	kN
1-64 UNIVERSITIES (See Colleges and Libraries)		
1-65 WAREHOUSES (See Storage)		
1-66 WORKROOMS, LIGHT without storage	2.5	1.8
1-67 WORKSHOPS (See Factories)		

2.105 Reduction in Total Imposed Floor Loads

2.105.1 Except as provided for in clauses 2.105.1 and 2.105.2 the reductions in assumed total imposed floor loads given in Table 2.105 may be taken in designing columns, piers, walls, their supports and foundations. For the purposes of clauses 2.105.1 to 2.105.3, a roof may be regarded as floor. For factories and workshops designed for 5 kN/m^2 or more, the reductions shown in Table 2.105 may be taken provided that the loading assumed is not less than it would have been if all floors had been designed for 5 kN/m^2 with no reductions.

TABLE 2.105

REDUCTION IN TOTAL DISTRIBUTED IMPOSED FLOOR LOADS

Number of floors including the roof carried by member under consideration	Reduction in total distributed imposed load on all floors carried by the member under consideration
	% (Percentage)
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

2.105.2 In the design of a beam or girder, where a single span of the beam or girder supports not less than 46 m^2 of floor at one general level, the imposed load may be reduced by 5 percent for each 46 m^2 supported, subject to a maximum reduction of 25 percent. This reduction or that given in Table 2.105, whichever is greater, may be taken into account in the design of columns or other types of members supporting such beams.

2.105.3 No reduction shall be made for any plant or machinery which is specifically allowed for, or for buildings for storage purposes, warehouses, garages and those office areas which are used for storage and filing purposes.

2.106 Imposed Roof Loads Other Than Wind Loads

2.106.1 INTERPRETATION - In clauses 2.106.2 to 2.106.11 inclusive, all roof slopes are measured from the horizontal, all loads are applied vertically and the 125 mm or 300 mm squares are measured on the roof slope.

2.106.2 FLAT ROOFS - On flat roofs and roofs not more than 10 degrees, where access is provided to the roof (other than for cleaning or repairs) and for all concrete roofs (with or without access) allowance shall be made for imposed load of 1.5 kN/m^2 measured on plan or a load of 1.8 kN concentrated on a square with 300 mm side.

2.106.3 On flat roofs and roofs not more than 10 degrees and of light weight construction where no access is provided to the roof (other than for cleaning and repairs), allowance shall be made for an imposed load of 0.75 kN/m^2 measured on plan, or a load of 0.9 kN concentrated on a square with a 300 mm side whichever produces the greater stress.

2.106.4 SLOPING ROOFS - On roofs with slope greater than 10 degrees where no access is provided (except for cleaning and repair) the following imposed loads shall be allowed:

(a) for a roof-slope of 30 degrees or less, 0.75 kN/m^2 measured on plan or a vertical load of 0.9 kN concentrated on a square with a 300 mm side, whichever produces the greater stress;

(b) for a roof-slope of 75 degrees or more, no allowances are necessary;

(c) for roof-slopes between 30 degrees and 75 degrees, the imposed load to be allowed for may be obtained by linear imterpolation between 0.75 kN/m^2 for a 30 degree roof-slope and zero imposed load for a 75 degree roof-slope.

2.106.5 CURVED ROOFS - The imposed load on a curved roof shall be calculated by dividing the roof into not less than five equal segments and then calculating the load on each, appropriate to its mean slope, in accordance with 2.106.2 to 2.106.4.

2.106.6 SPECIAL PURPOSE ROOFS AND ROOFS PROVIDING VANTAGE POINTS FOR CROWDS

2.106.7 When used for incidental promenade purposes, or when likely to be used as a vantage point (for example, adjacent to sports grounds and processional routes) roofs shall be designed for a minimum live load of 3.0 kN/m^2.

2.106.8 When designed for roof-garden or assembly uses minimum live load shall be 5.0 kN/m^2.

2.106.9 Roofs to be used for other special purposes shall be designed for appropriate loads as directed or approved by the Building Official or competent authority.

2.106.10 LOAD REDUCTION ON ROOFS - Imposed loads on members with tributary areas exceeding 46 m^2 may be reduced in accordance with clause 2.105.2.

2.106.11 ROOF COVERINGS - To provide for loads incidental to maintenance, all coverings (other than glazing) at a slope of less than 45 degrees shall be capable of carrying a load of 0.9 kN concentrated on any square with a 125 mm side.

2.107 Dynamic Loading

2.107.1 Where loads arising from machinery, runways, cranes, fork-lift trucks and other plants producing dynamic effects are supported by or communicated to the framework, allowance shall be made for these dynamic effects, including impact, by increasing the dead weight values by an adequate amount. In order to ensure due economy in design, the appropriate dynamic increase for all members affected shall be ascertained as accurately as possible.

2.107.2 For crane gantry girders, the following allowances shall be deemed to cover all forces set up by vibration, shock from slipping of slings, kinetic action of acceleration and retardation and impact of wheel loads:

(a) For loads acting vertically, the maximum static wheel loads shall be increased by:
(1) 25% for an electric overhead crane; or
(2) 10% for a hand-operated crane.

(b) The horizontal force acting transverse to the rails shall be taken as the following percentage of the combined weight of the cab and the load lifted:
(1) 10% for an electric overhead crane; or
(2) 5% for a hand-operated crane.
This force shall be taken into account when considering the lateral rigidity of the rails and their fastenings.

(c) Horizontal forces acting along the rails shall be taken as 5 per cent of the static wheel loads which can occur on the rails, for overhead cranes whether electric or hand-operated.

2.107.3 The forces specified in either clause 2.107.2(b) or 2.107.2(c) shall be considered as acting at the rail level and being appropriately transmitted to the supporting systems.

2.107.4 Gantry girders and their vertical supports shall be designed on the assumption that either of the horizontal forces in clause 2.107.2(b) or 2.107.2(c) may act at the same time as the vertical load.

2.107.5 2.107.2 applies only to single crane operation and to simple forms of crane gantry construction. For heavy cranes, high-speed operation cranes, or multiple cranes on a single-gantry, special arrangements shall be made.

2.108 Parapets, Balustrades and Ornamental Projections

2.108.1 PARAPETS AND BALUSTRADES - Parapets and balustrades shall be designed for the minimum loads given in Table 2.108. These are expressed as horizontal forces acting at handrail or coping level.

2.108.2 ORNAMENTAL PROJECTIONS - Ornamental projections shall be designed for a minimum load of 0.30 kN/m. See Table 2.108 which is expressed as a vertical force acting at the extremity of such projections.

TABLE 2.108

LOADS ON PARAPETS, BALUSTRADES AND ORNAMENTAL PROJECTIONS

Use	Intensity of Horizontal Loads	Intensity of Vertical Loads
	kN/m run	kN/m
Light access stairs, gangways and the like not more than 600 mm wide	0.220	
Light access stairs, gangways and the like, more than 600 mm wide, stairways, landings and balconies, private and domestic	0.360	
All other stairways, landings and balconies, and all paraptets and handrails to roof	0.740	
Panic barriers	0.30	
Ornamental projections		0.30

2.109 Vehicle Barriers for Car Park

2.109.1 Where a barrier to withstand the force of a vehicle is required for a car park, it shall be designed in accordance with the following clauses:

The barrier shall withstand a force F uniformly distributed over any length of 1.5 m where

$$F = \frac{(1/2\ mv^2)}{(c + b)}\ \text{kN};$$

m = mass of vehicle in kg;
v = velocity in m/s;
c = deflexion of the vehicle in mm; and
b = deflexion of the barrier in mm.

2.109.2 Where the car park has been designed on the basis that vehicles using it will not exceed 2500 kg the following values shall be used to determine the force F:

m = 1500 kg*
V = 4.47 m/s; and
c = 100 mm unless better evidence is available.

2.109.3 For a rigid barrier the force appropriate to vehicles up to 2500 kg shall be taken as 150 kN.

2.109.4 Where the car park has been designed for vehicles exceeding 2500 kg the following values shall be used to determine the force F:

m = the actual mass of the vehicle for which the car park is designed in kg;
v = 4.47 m/s; and
c = 100 mm unless better evidence is available.

2.109.5 The impact force provided under clauses 2.109.2 to 2.109.4 shall be considered to act at bumper height. In the case of car parks intended for motor cars not exceeding 2500 kg this shall be taken as 375 mm above the floor level.

2.109.6 Barriers to access ramps of car parks shall be designed to withstand one-half** of the force determined in clauses 2.109.2 to 2.109.4 acting at a height of 610 mm above the ramp.

2.109.7 Where a straight ramp for downward travel is more than 20 m in length, the barrier opposite the lower end of the ramp shall be designed to withstand twice the force determined as in clauses 2.109.2 to 2.109.4 acting at a height of 610 mm above the ramp surface.

2.109.8 The recommendations in clauses 2.109.2 to 2.109.4 may be used to form the basis of design either within or beyond the usual service ability limits of water.

* The mass of 1500 kg is taken as being more representative of the vehicle population than the extreme value of 2500 kg.

** The force recommended in clause 2.109.6 is only half of that recommended in clauses 2.109.2 to 2.109.4 because although the speed of vehicles may be greater the angle of impact is likely to be less. At the ends of straight ramps however, not only is the speed likely to be greater but the angle of impact will also be greater, so that the barrier must withstand a greater force, and the force recommended in clause 2.109.6 is therefore double that given in clauses 2.109.2 to 2.109.4.

FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

Feet^2 (ft^2) x 0.0929030	= Meters^2 (m^2)
Acres x 4046.86	= Meters^2 (m^2)
Miles^2 (mi^2) x 2.58999	= Kilometers^2 (km^2)

VOLUME

Feet^3 (ft^3) x 0.02831685	= Meters^3 (m^3)
Feet^3 (ft^3) x 28.31685	= Liters**
Yards^3 (yd^3) x 0.764555	= Meters^3 (m^3)
Acre feet x 1233.48	= Meters^3 (m^3)
Gallons (gal) x 0.00454609	= Meters^3 (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

Feet^3 per second (ft^3/s) x 0.02831685	= Meters^3 per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= Meters^3 per second (m^3/s)
Acre-feet per day x 0.0142764	= Meters^3 per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= Meters^3 per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 ($lb\text{-}s/ft^2$) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

---

* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 3

Structural Design Requirements
EARTHQUAKE LOAD

1985

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 3
EARTHQUAKE LOAD

Caribbean Community Secretariat
Georgetown
Guyana

1985

PART 2

SECTION 3

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TABLES

FOREWORD

Natural disasters such as earthquakes and hurricanes cause considerable damage in the West Indies each year. Recognizing this, in 1969 a seismic code committee of the Association of Professional Engineers of Trinidad and Tobago (APETT) was formed to develop a suitable code of practice for earthquake resistant design of structures and in February 1970 a preliminary report was published for comment. Subsequent work of the committee resulted in the conclusion that the entire Caribbean region stretching from Trinidad and Tobago in the south to Jamaica in the north and including all the other Eastern Caribbean Islands all corresponded and in terms of frequency of events per century of MM intensity equal or greater than VI, to the highest risk zone (Zone 3) of California, U.S.A.

In their final draft code of practice therefore the then Seismic Code Committee recommended that the design of all buildings in the Caribbean should be based on the "Recommended Lateral Force Requirements and Commentary" of the Seismology Committee of the Structural Engineers Association, California 1968 (known as SEAOC Code).

At about the same time practising engineers in the Caribbean region were giving much attention to the development of a regional unified building code for use within the various West Indian islands. Towards this end, two regional symposia sponsored by the Council of Caribbean Engineering Organizations (CCEO) were held in 1973 and 1974 in Jamaica with the main objective of finalizing such a code. At the 1973 symposium, some progress was made towards this goal when certain codes of practice were adopted by the CCEO for regional use amongst which was the draft seismic code prepared by APETT in July 1972. It was intended that these codes of practice would eventually be incorporated into the proposed regional Caribbean Uniform Building Code (CUBiC) under a grant agreement between USAID and CARICOM Secretariat.

However, since the 1968 edition of the the SEAOC code, there have been many changes culminating in the 1975 edition which included several major changes in the base shear formula V = ZIKCSW. The two main coefficients are the I and S coefficients which take into account the importance of the structure and the site structure resonance respectively. The implications of these changes, in general, were to produce increases in the required lateral forces with a consequent increase in the construction cost of buildings. In view of these changes, practising engineers in the Caribbean felt it was opportune to re-examine the whole question of seismic design in the West Indies and in this connection, the first Caribbean Conference on Earthquake Engineering was held from January 9-12th 1978 in Trinidad with the main objectives of obtaining a consensus of what levels of lateral forces should be used in the design of structures in the West Indies and hence of producing a revised code of practice for earthquake resistant design.

One of the main recommendations arising out of the Conference was that a Seismic Code Drafting Committee should be formed to review the main points of the Conference and to undertake the task of finalizing the revision of the draft seismic code. Such a committee was formed shortly after the conference and at their first meeting held in Barbados on June 30, 1978, it was recommended that the "Recommended Lateral Force Requirements and Commentary" of the Structural Engineers Association of California 1975 Edition (SEAOC 1975) should be adopted in principle to form the basis of the revised West Indian Seismic Code and values of the Zonal coefficient Z in the base shear formula V = ZIKCSW were given for various West Indian islands.

A follow-up seminar was held in February 1983 and a review of the Z values was undertaken in the light of more recent seismic data.

In November 1983, the Short Term Consultant Principia Mechanica of London, England, reviewed these zonal coefficients and also reviewed the seismic codes of various countries in order to develop appropriate clauses for CUBiC. It was concluded that some sections of SEAOC, UBC, ATC and New Zealand Codes were relevant and appropriate to CUBiC. Accordingly, the seismic code provisions of CUBiC are based essentially on SEAOC but with appropriate sections from UBC, ATC, and New Zealand Codes.

It must be stressed that the recommendations given herein are based on current knowledge of the seismicity in the Caribbean and that the clauses should be continually reviewed in the light of any further research in this field.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

The first number indicates the Part of the Code, the first digit in the second number indicates the Section in the Part, the second and third digits in the second number indicate the sub-section in the Section, and the third number indicates the Article in the sub-section. These are illustrated as follows:

2	Part 2
2.500	Part 2, Section 5
2.506	Part 2, Section 5, sub-section 6
2.506.3	Part 2, Section 5, sub-section 6, Article 3

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

PART 2

SECTION 3

2.300 EARTHQUAKE LOAD

2.301 Scope

This Code contains Seismic Design provisions recommended for use in the Design and Construction of buildings in the West Indies. These provisions establish requirements for the Design and Construction of new buildings to resist the effects of earthquake motions.

2.301.1 These provisions do not cover requirements for Design and Construction of special structures including, but not limited to, bridges, transmission towers, industrial towers and equipment, piers and wharves, hydraulic structures, off-shore structures and nuclear reactors. These special structures require special consideration of their response characteristics and environment which is beyond the scope of these provisions.

2.302 Definitions

Base is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported.

Box System is a structural system without a complete vertical load carrying space frame. In this system, the required lateral forces are resisted by shear walls or braced frames as hereinafter defined.

Braced Frame is a truss system or its equivalent which is provided to resist lateral forces in the frame system and in which the members are subjected primarily to axial stresses.

Ductile Moment Resisting Space Frame is a moment resisting space frame complying with the requirements given in clauses 2.307.4 and 2.307.6.

Essential Facilities are those structures which must be functional for emergency post earthquake operations.

Lateral Force Resisting System is that part of the structural system assigned to resist the lateral forces prescribed in clause 2.305.3.

Moment Resisting Space Frame is a vertical load carrying space frame in which the members and joints are capable of resisting forces primarily by flexure.

Shear Wall is a wall designed to resist lateral forces parallel to the plane of the wall.

Space Frame is a three dimensional structural system, without bearing walls, composed of interconnected members laterally supported as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

Vertical Load Carrying Space Frame is a space frame designed to carry all vertical loads.

2.303 **Symbols and Notations**

The following symbols and notations apply to the provisions of this Section:

C	Numerical coefficient as specified in Section 2.305.5
C_p	Numerical coefficient as specified in Section 2.305.15 and as set forth in Table 2.305.3
D	The dimension of the building in feet, in a direction parallel to the applied forces
$\delta_i\ \delta_n$	Deflections at levels i and n respectively, relative to the base, due to applied lateral forces.
F_i, F_n, F_x	Lateral force applied to level i,n, or x, respectively.
F_p	Lateral forces on a part of the structure and in the direction under consideration.
F_{xm}	The portion of the seismic base shear, V_m induced at level x.
F_t	That portion of V considered concentrated at the top of the structure in addition to F_n.
g	Acceleration due to gravity.
h_i, h_n, h_x	Height in feet above the base to level i,n, or x respectively.
I	Occupancy importance coefficient as set forth in Section 2.305.6.
K	Numerical coefficient as set forth in Table 2.305.2.

Level i	Level of the structure referred to by the subscript i. i = 1 designates the first level above the base.
Level n	That level which is uppermost in the main portion of the structure.
Level x	That level which is under design consideration. x = 1 designates the first level above the base.
N	The total number of stories above the base to level n.
S	Numerical coefficient for site-structure resonance.
T	Fundamental elastic period of vibration of the structure in seconds in the direction under consideration.
T_m	The modal period of vibration of the m^{th} mode of the building.
T_s	Characteristic site period.
V	The total lateral force or shear at the base.
W	The total dead load and applicable portions of other loads.
w_i, w_x	That portion of W which is located at or is assigned to level i or x respectively.
w_{px}	The weight of the diaphragm and the elements tributary thereto at level x, including 25 percent of the floor live load in storage and warehouse occupancies.
W_p	The weight of a portion of a structure.
Z	Numerical coefficient related to the seismicity of a region as set forth in Table 2.305.1.

2.304 **Methods of Analysis**

2.304.1 Buildings shall be analysed by the equivalent static force method specified in sub-section 2.305 or by dynamic analysis, specified in sub-section 2.306.

2.304.2 Buildings which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent storeys, or other unusual features shall be analysed by dynamic methods. In particular, buildings classified in Importance Groups I and II (refer to clause 3.205.6) shall be analysed by dynamic methods when:

(a) the seismic force resisting system does not have the same configuration in all storeys and in all floors.

(b) The floor masses differ by more than 30% in adjacent floors.

(c) The cross-sectional areas and moments of inertia of structural members differ by more than 30% in adjacent stories.

2.304.3 Nothing in these recommendations shall be deemed to prohibit the submission of properly substantiated technical data for establishing the lateral design forces and distribution by dynamic analysis.

2.305 Equivalent Static Force Analysis

2.305.1 GENERAL - The horizontal seismic forces specified in this section shall be applied simultaneously at each floor and roof level.

2.305.2 For buildings symmetrical about at least one axis and for buildings with seismic resistance elements located along two perpendicular directions, the specified force may be assumed to act separately along each of these two horizontal directions. For other buildings, different directions of application of the specified forces shall be considered so as to produce the most unfavourable effect in any structural element.

2.305.3 EQUIVALENT STATIC FORCES - Except as provided in clauses 2.304.2 and 2.304.3, any structure shall be designed and constructed to resist minimum total lateral seismic forces assumed to act nonconcurrently in the direction of the main axis of the structure in accordance with the formula:

$$V = ZCIKSW \qquad (1)$$

2.305.4 The value of the zonal coefficient, Z, shall be given in Table 2.305.1.

2.305.5 The value of C shall be determined in accordance with the formula:

$$C = \frac{1}{15\sqrt{T}} \qquad (2)$$

where C need not exceed 0.12.

The period T shall be established using the structural properties and deformational characteristics of resisting elements in a properly substantiated analysis such as the formula:

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n-1} f_i \delta_i}} \qquad (3)$$

where the values of f_i represent the lateral force distribution in accordance with clause 2.305.11. In the absence of a period determination as indicated above, the value of T for buildings may be determined by one of the following formulae:

For moment resisting structures where the frames are not enclosed or adjoined by more rigid components tending to prevent the frames from deflecting when subjected to seismic forces:

$$T = C_T h_n^{3/4} \qquad (4)$$

where: C_T = 0.35 for steel frames
C_T = 0.025 for concrete frames
h_n = the height in feet above the base to the highest level of the building.

For all other buildings:

$$T = \frac{0.05 h_n}{\sqrt{L}} \qquad (5)$$

where: L = the overall length (in feet) of the building at the base in the direction under consideration.

2.305.6 The importance factor, I, shall be determined as follows:

Class I Buildings: I = 1.5

These are essential facilities required for use in the aftermath of a major earthquake, e.g. hospitals, fire stations, communication centres etc.

Class II Buildings: I = 1.2

These are public buildings and buildings which accommodate large numbers of people, e.g. cinemas, theatres, schools, defence establishments etc.

Class III Buildings: $I = 1.0$

All other buildings not included in Class I or Class II above.

2.305.7 The structural behaviour factor shall not be less than that given in Table 2.305.2

2.305.8 The value of the soil factor, S, shall be determined by the following formulae but shall not be less than 1.0:

For $\frac{T}{T_s} = 1.0$ or less, $S - 1.0 + \frac{T}{T_s} - 0.5 \left(\frac{T}{T_s}\right)^2$ (6)

For $\frac{T}{T_s}$ greater than 1.0,

$$S = 1.2 + 0.6 \frac{T}{T_s} - 0.3 \left(\frac{T}{T_s}\right)^2 \qquad (7)$$

T shall be established by a properly substantiated analysis but T shall not be taken as less than 0.3 seconds.

The range of values of T_s may be established from properly substantiated geological data, except that T_s shall not be taken as less than 0.5 seconds nor more than 2.5 seconds. T_s shall be that value within the range of site periods as determined above, that is nearest to T.

Where T has been established by a properly substantiated analysis and exceeds 2.5 seconds, the value of S may be determined by assuming a value of 2.5 seconds for T_s.

2.305.9 W is the total gravity load of the buildings. W shall be taken equal to the total weight of the structure and applicable portions of other components, including but not limited to, the following:

(a) Partitions and permanent equipment including operating contents.

(b) For storage and warehouse structures, a minimum of 25% of the floor live load.

2.305.10 The product CS need not exceed 0.14.

2.305.11 VERTICAL DISTRIBUTION OF LATERAL FORCES - The lateral seismic shear force, F_x, induced at any level, shall be determined in accordance with the following formula:

$$F_x = C_{vx}V \qquad (8)$$

where $$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_j h_j^k} \qquad (9)$$

k is an exponent related to the building period as follows:

For buildings having a period of 0.5 seconds or less,
k = 1
For buildings having a period of 2.5 seconds or more,
k = 2
For buildings have a period between 0.5 and 2.5 seconds,
k may be 2

w_j, w_x = the portion of W located at or assigned to level j or x.
h_j, h_x = the height above the base at level j or x.

2.305.12 OVERTURNING MOMENTS - The seismic overturning moments shall be derived from the distribution of the horizontal seismic forces according to clause 2.305.11 without reduction.

2.305.13 DISTRIBUTION OF HORIZONTAL SHEAR - Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be part of the lateral force-resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design.

2.305.14 HORIZONTAL TORSIONAL MOMENT - Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the centre of mass and the centre of rigidity. Negative torsional shears shall be neglected. Where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the storey shear acting with an eccentricity of not less than five percent of the maximum dimension at that level.

2.305.15 LATERAL FORCE ON ELEMENTS OF STRUCTURES - Parts or portions of structures and their anchorage to the main structural system shall be designed for lateral forces in accordance with the formula:

$$F_p = ZIC_pW_p \qquad (10)$$

2.305.16 The values of Cp are set forth in Table 2.305.2. The value of the I coefficient shall be the value used for the building; except when considering life safety systems in which case I shall be taken as 1.5.

2.305.17 DRIFT PROVISIONS - Lateral deflections or drift of a storey relative to its adjacent stories shall not exceed 0.005 times the storey height unless it can be demonstrated that greater drift can be tolerated. The displacement calculated from the application of the required lateral forces shall be multiplied by 1.0/K to obtain the drift. The ratio 1.0/K shall not be less than 1.0.

2.306 Dynamic Analysis

2.306.1 GENERAL - Dynamic analysis shall be performed for buildings which do not satisfy the conditions for the application of equivalent static methods (refer to sub-section 2.304).

2.306.2 Dynamic analysis may be any standard procedure including modal analysis and direct integration.

2.306.3 When direct integration methods are used, special attention shall be given to the selection of the time-step of integration in accordance with recommendations given in standard textbooks.

2.306.4 When modal response spectrum analysis is used, the recommendations given in clauses 2.306.5 to 2.306.13 shall be followed.

2.306.5 MODAL ANALYSIS PROCEDURE - The building may be modelled as a system of masses lumped at the floor levels, with each mass having one degree of freedom, that of lateral displacement in the direction under consideration.

2.306.6 The analysis shall include, for each of two mutually perpendicular axes, at least the lowest three modes of vibration with periods greater than 0.4 seconds, whichever is greater, except that for structures less than three stories in height, the number of modes shall equal the number of stories.

2.306.7 The required period and mode shapes of the building in the direction under consideration shall be calculated by established methods of mechanics for the fixed base condition using the masses and elastic stiffness of the seismic resisting system.

2.306.8 The portion of the base shear contributed by the m^{th} mode, V_m, shall be determined in accordance with the formula:

$$V_m = ZC_mKISW_m \qquad (11)$$

where

$$C_m = \frac{1}{15\sqrt{T_m}} \qquad (12)$$

the subscript, m, refers to the m^{th} mode. T_m shall be computed in accordance with clause 2.306.7. W_m is the effective modal gravity load determined in accordance with the following formula:

$$W_m = \frac{\sum_{j=1}^{n} w_j v_{im}}{\sum_{j=1}^{n} w_j v_{im}} \qquad (13)$$

where v_{im} = the displacement amplitude at the i^{th} level of the building when vibrating in its m^{th} mode.

The product C_mS need not exceed 0.14.

2.306.9 The modal force at each level, F_{xm}, shall be determined in accordance with the formula:

$$F_{xm} = C_{vxm}V_m \qquad (14)$$

where

$$C_{vxm} = \frac{w_x \; v_{xm}}{\sum_{i=1}^{n} w_i v_{im}} \qquad (15)$$

2.306.10 The modal drift in each storey shall be determined from the modal displacement u_{xm}, which shall be given by:

$$U_{xm} = \frac{g\, T_m^{\,2}\, F_{xm}}{4\pi^2\, W_x} \qquad (16)$$

2.306.11 The shear forces and overturning moments in walls and braced frames at each level shall be determined using linear static methods with the seismic forces and displacements as determined from clauses 2.306.9 and 2.306.10.

2.306.12 The design value for base shear, each of the storey shear, moment and drift quantities and the deflection at each level shall be determined by combining their modal values. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values. If two or more modal periods are within 10% of each other, these modes shall be summed absolutely and the resultant added to the other modes using the square root of the sum of the square procedure.

2.306.13 The base shear computed from the modal combination procedure shall not be less than 75% of the value computed by the equivalent static method.

2.307 Design Principles

2.307.1 SYMMETRY - The main elements of a building that resist seismic forces shall, as nearly as is practicable, be located symmetrically about the centre of mass of the building.

2.307.2 DUCTILITY - The building as a whole, and all of its elements that resist seismic forces or moments, or that in case of failure are a risk to life, shall be designed to possess ductility.

2.307.3 Structural systems intended to dissipate seismic energy by ductile flexural yielding shall have all primary elements resisting seismic forces detailed in accordance with special requirements for ductile detailing in the appropriate material code.

2.307.4 In Seismic Zones No. 2, No. 3 and No. 4, all concrete space frames required by design to be part of the lateral force-resisting system and all concrete frames located in the perimeter line of vertical support shall be ductile moment resisting space frames.

EXCEPTION: Frames in the perimeter line of the vertical support of the buildings designed with shear walls taking 100 percent of the design lateral forces need only conform with clause 2.307.5.

2.307.5 In Seismic Zones No. 2, No. 3 and No. 4, all framing elements not required by design to be part of the lateral force-resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity and induced moment due to 3/K times the distortions resulting from the code-required lateral forces. The rigidity of other elements shall be considered in accordance with clause 2.305.13.

2.307.6 Moment-resisting space frames and ductile moment-resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

2.307.7 Buildings designed for flexural ductile yielding, or for yielding in diagonal braces, shall be the subject of capacity design. In the capacity design of earthquake resistant structures, energy dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained throughout the deformations that may occur.

2.307.8 Columns or walls, including their joints and foundations, which are part of a two-way horizontal force resisting system, shall be designed for the concurrent effects resulting from the simultaneous yielding of all beams or diagonal braces framing into the column or wall from all directions at the level under consideration and as appropriate at other levels.

2.307.9 Ductile frames shall be capable of dissipating seismic energy in a flexural mode at a significant number of beam hinges, except that dissipation of seismic energy in column hinges is permitted for buildings which comply with clauses 2.307.13 and 2.307.14.

2.307.10 Non-ductile failure in beams shall be avoided. The yeild capacities of beams shall be assessed for the actual material quantities in the beam and any adjacent portions of slabs that are likely to be strained simultaneously.

2.307.11 Failure of beam-column junction zones and the formation of plastic hinges in beam-column junction zones shall be avoided.

2.307.12 Non-ductile failure of columns shall be avoided. Columns shall be designed to have adequate over-capacity to avoid the formation of hinges and column hinge mechanisms, except as permitted by clauses 2.307.13 and 2.307.14. Column capacities shall be sufficient to allow for the following:

(a) Inelastic effects leading to a distribution of beam capacity moments into columns different from the distribution derived from elastic analysis;

(b) Column axial loads appropriate to the simultaneous formation of beam hinges in several storeys.

2.307.13 In 'adequately redundant' structures the formation of a column hinge because of bending moment and axial tension, or bending moment and low axial compression, is permissible proided the sear capacity of the column is maintained. For the purposes of this clause a structure may be considered 'adequately redundant' if for every column with a potential plastic hinge at least three other columns of the main horizontal load resisting system, interconnected by a rigid diaphragm and within the same storey height, can be shown to remain elastic under the capacity design horizontal load.

2.307.14 In single or two-storey structures and in the top storey of a multi-storey building, column hinge mechanisms are permitted.

2.307.15 For ductile coupled shear walls, the requirements for clause 2.307.17, regarding height to width ratio and minimum width may be departed from provided the vertical members are detailed as required for the columns of ductile frames.

2.307.16 For ductile complex shear walls, the design shall be such that the coupling beams yield before the walls do, and the coupling beams shall be so proportioned as to dissipate a significant proportion of seismic energy.

2.307.17 Ductile cantilever shear walls shall be suitably designed and detailed to ensure that energy dissipation will be by ductile flexural yielding and that the wall will not fail prematurely in a non-ductile manner. The height to width ratio shall not be less than Z, and the width shall no be less than 5 feet, and any openings shall be such as not to have a significant effect on the behaviour of the wall under earthquake attack.

2.307.18 Shear walls not designed for ductile flexural yielding but having the ability to dissipate a signifcant amount of seismic energy shall be suitably detailed to ensure:

(a) Adequate confinement of concrete at potential hinges to provide limited ductile flexural yielding, and

(b) That in walls, with height to width ratio less than or equal to 2, under earthquake attack a distributed system of cracking of controlled width will form so as to preclude premature shear failure.

2.307.19 REINFORCED MASONRY AND CONCRETE - All elements within structures located in Seismic Zones No. 2, No. 3 and No. 4 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete. Principal reinforcement in masonry shall be spaced 2 feet maximum on centre in buildings using a moment-resisting space frame.

2.307.20 ANCHORAGE OF CONCRETE OR MASONRY WALLS - Concrete or masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter or a minimum force of 200 pounds per lineal foot of wall, whichever is greater. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall.

2.307.21 WOOD DIAPHRAGMING PROVIDING LATERAL SUPPORT FOR CONCRETE AND MASONRY WALLS - Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to clause 2.307.20. In Zones No. 2, No. 3 and No. 4, anchorage shall not be accomplished by the use of toenails or nails subjected to withdrawals, nor shall wood framing be used in cross-grain bending or cross-grain tension.

2.307.22 PILE CAPS AND CAISSONS - Individual pile caps and caissons of every building shall be interconnected by ties, each of which can carry by tension and compression a minimum horizontal force equal to 10 percent of the larger pile cap or caisson loading, unless it can be demonstrated that equivalent restraint can be provided by other approved methods.

2.307.23 EXTERIOR ELEMENTS - Precast or prefabricated nonbearing, non-sear wall panels or similar elements which are attached to or enclose the exterior shall be designed to resist the forces determined from Formula 10 in clause 2.305.15 and shall accomodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other similar elements shall be supported by means of cast-in-place concrete or mechanical connections and fasteners in accordance with the following provisions:

> Connections and panel joints shall allow for a relative movement between storeys of not less than two times storey drift caused by wind or (3.0/K) times the calculated elastic storey displacement caused by required seismic forces, of 1/2 inch, whichever is greater. Connections to permit movement in the plane of the panel for storey drift shall be properly designed sliding connections using slotted or oversized holes or may be connections which permit movement by bending of steel or other connections providing equivalent sliding and ductility capacity.

Bodies of connectors shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete of brittle failures at or near welds. The body of the connector shall be desgined for one-and-one-third times the force determined by Formula 10 in clause 2.305.15. Fasteners attaching the connector to the panel or the structure such as bolts, inserts, welds, dowels etc., shall be designed to insure ductile behaviour of the connector or shall be designed for four times the load determined from Formula 10.

Fasteners embedded in concrete shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

The value of the coefficient I shall be 1.0 for the entire connector assembly in Formula 10.

TABLE 2.305.1
Z VALUES

Territory	Z Value
Jamaica	.75
Antigua	.75
St. Kitts-Nevis	.75
Montserrat	.75
Dominica	.75
St. Lucia	.75
St. Vincent	.50
Grenada	.50
Barbados	.375
NW Trinidad	.75
Rest of Trinidad	.50
Tobago	.50
Guyana Essequibo	.25
Rest of Guyana	.00
Belize - areas within 100 km of southern border i.e. including San Antonio and Punta Gorda but excluding Middlesex, Pomona and Stann Crecil	.75
- Rest of Belize	.50

TABLE 2.305.2
RECOMMENDED K FACTOR

A. VALVES FOR STEEL AND CONCRETE

Item	Description	K*
1	Ductile frames with an adequate number of possible plastic beam hinges	0.8
2	Ductile frames with an inadequate number of possible plastic beam hinges	1.0
3	Ductile coupled shear walls	0.8
4	Two or more parallel and approximately symmetrically arranged cantilever shear walls	1.0
5	Single ductile cantilever shear walls	1.2
6	Shear walls not designed for ductile flexural yielding but having the ability to dissipate a significant amount of seismic energy	1.6
7	Buildings with diagonal bracing capable of plastic deformation in tension only	
	a) Single storey	2.0
	b) Two or three storeys	2.5 or by special study
	c) More than three storeys	by special study
8	a) Buildings in which part of the horizontal load is resisted by item 7 bracing and part by an item 1 or item 2 frame	1.6 or by special study
8	b) Buildings with diagonal bracing capable of plastic deformation in both tension and compression	1.6 or by special study
9	Small tanks on the ground	2.0

* For steel these values to be multiplied by 0.8
For prestressed concrete these values to be multiplied by 1.2
For reinforced masonry these values to be multiplied by 1.2

TABLE 2.305.2 (continued)

B. VALUES FOR TIMBER

Item	Description	K
B1	Shear walls or diaphragm	
	(a) Ductile	1.0
	(b) Ductile and stiffened with elastomeric adhesive	1.0
	(c) Limited ductility fixed with elastomeric adhesive	1.2
B2	Moment resisting frames	
	(a) Ductile with an adequate number of possible plastic beam hinges	1.2
	(b) Ductile with an inadequate number of possible plastic beam hinges	1.5
	(c) As for item B2(a) but with connections of limited ductility	1.5
	(d) As for item B2(b) but with connections of limited ductility	1.7
	(e) Non-ductile	2.4
B3	Diagonally braced with timber members capable of acting as struts or ties:	
	(a) With ductile end connections	1.7
	(b) With end connections having limited ductility	2.0

TABLE 2.305.3
HORIZONTAL FORCE FACTOR "C_p" FOR ELEMENTS OF STRUCTURES

Part or Portion of Buildings	Horizontal Direction of Force	Value of C_p (1)
Cantilevered Elements a. Parapets	Normal to flat surfaces	.8
b. Chimneys of stacks	Any direction	
All other walls, partitions and similar elements	Any direction	.3
Exterior and interior ornamentations and appendages	Any direction	.8
When connected to, part of, or housed within a building: a. Penthouses, anchorage and supports for tanks including contents, chimneys and stacks b. Storage racks plus contents c. Suspended ceilings (3) d. All equipment or machinery	Any direction	.3(2)(4)
Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly	Any direction	.3(4)

(1) Cp for elements laterally self supported only at ground level may be 2/3 of the value shown.

(2) For flexible and flexibly mounted equipment and machinery, the appropriate values of Cp shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed but shall not be less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery.

For Essential Facilities and life safety systems, the design and detailing of equipment which must remain in place and be functional following a major earthquake shall consider the effect of drift.

(3) Ceiling weight shall include all light fixtures and other equipment or partitions which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot shall be used.

(4) The force shall be resisted by positive anchorage and not by friction.

Reference Publications

This Code refers to the following publications:

1. "Recommended Lateral Force Requirements and Commentary" of the Structural Engineers Association of California. Latest Edition (SEAOC).

2. Uniform Building Code. Latest Edition.

3. Tentative Provisions for the Development of Seismic Regulations for Buildings. Applied Technology Council (ATC) Publications ATC3-06 NBS-SP-510.

4. Code of Practice for the Design of Concrete Structures. Standards Association of New Zealand NZS 3101. Part 1, 1982.

5. NZS 4203: 1976 - Code of Practice for general Structural Design and Design loadings for buildings. Standards Association of New Zealand.

FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

Feet^2 (ft^2) x 0.0929030	= Meters^2 (m^2)
Acres x 4046.86	= Meters^2 (m^2)
Miles^2 (mi^2) x 2.58999	= Kilometers^2 (km^2)

VOLUME

Feet^3 (ft^3) x 0.02831685	= Meters^3 (m^3)
Feet^3 (ft^3) x 28.31685	= Liters**
Yards^3 (yd^3) x 0.764555	= Meters^3 (m^3)
Acre feet x 1233.48	= Meters^3 (m^3)
Gallons (gal) x 0.00454609	= Meters^3 (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

Feet^3 per second (ft^3/s) x 0.02831685	= Meters^3 per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= Meters^3 per second (m^3/s)
Acre-feet per day x 0.0142764	= Meters^3 per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= Meters^3 per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 (lb-s/ft^2) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

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* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 4
BLOCK MASONRY

Caribbean Community Secretariat
Georgetown
Guyana

1985

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 4

Structural Design Requirements
BLOCK MASONRY

1985

PART 2

SECTION 4

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FOREWORD

In developing this Section 4 on "Requirements for Masonry Building Design and Construction", a decision was made that earlier contributions by Caribbean engineers towards the development of CUBiC would be improved and incorporated.

"Tentative Recommendations for Design and Construction of Load Bearing Concrete Blockwork", 1975, produced by the Jamaica Institution of Engineers for the Council of Caribbean Engineering Organisations, was used as the basis of Section 4A. This Section deals with concrete blockwork subjected to moderate to heavy working stresses in areas of moderate to low seismicity. However, although it is proposed for use with Part 2 Section 3 - "Earthquake Loads" for earthquake resistant design, it does not include recommendations for construction standards to cover other ranges of seismicities.

"Tentative Provisions for the Development of Seismic Regulations for Buildings: (ATC-3) published by Applied Technology Council, U.S.A., was used as the basis for Section 4B, which provides a rational method of varying earthquake resistant provisions in buildings, in accordance with the varying seismicities of different countries in the region. This latter document represents the latest Code recommendations on the subject in the U.S.A. and it has therefore not yet gained universal acceptance in that country. Its methodology was selected for Section 4 because it proved to be uniquely appropriate for masonry design in the scattered islands of the Caribbean.

The results of "shaking table" studies done at the University of California, and Guidelines for One Storey Masonry Houses in Seismic Zone 2 by ATC have encouraged the appeal by the region's engineers and housing officials, to reduce the minimum percentages of reinforcement recommended by Codes. Section 4C has used this work, along with subsequent Caribbean adaptations, to present recommendations for single storey reinforced masonry buildings, with light roofs and for other buildings in areas of low seismicity. Variuous other types of traditional construction, particularly those using indigenous materials, are dealt with in Appendix 1 which permits the user to bypass the more complex Sections, 4A and 4B entirely, for appropriate categories of buildings. Model Regulations for Small Buildings in Earthquake and Hurricane areas by the Building Research Station, UK have been used as a basis for this aspect.

Acknowledgement is made of many other reference sources which are listed in Appendix 3.

SCOPE AND GUIDE TO USE OF SECTION 4

These provisions for the Design of Concrete Masonry are divided into the following:

Section 4A
Section 4B
Section 4C
Appendix 1

The various sections are used for different seismic performance categories* of buildings as follows:

SECTION 4A - provides basic design requirements which are considered appropriate for masonry subjected to moderate to heavy working stresses in areas of low to moderate seismicity. It is specifically recommended for use in designing buildings:

(a) Over 10.5 m high in Seismic Performance category A.
(b) In category B, provided that the special material and construction limitations in sub-section 2.426 Section 4B are considered.

The Section is appropriate for design for allowable stresses, assuming elastic behaviour, and using seismic forces derived from the recommendations of Part 2 Section 3 for earthquake considerations.

Alternatively, where ultimate loads are considered (e.g. factored Part 2, Section 3 loads or loads derived from Section 4B), Strength Design Methods, may be used, with ∅ factors as set out in Section 4B.

SECTION 4B - provides additional requirements for buildings in seismic categories B, C and D over and above those of Section 4B.

∅ factors are given for strength design and material strengths are based on 2.5 times the allowable stresses set out in Section 4A.

SECTION 4C - gives requirements for:

(a) single storey buildings with light roofs in all territories (i.e. seismic performances categories A to C) except for the case of essential facilities.

(b) buildings up to 10.6 m high in seismic performance category A (i.e. areas of low seismicity).

TABLE 2.4.1 - Illustrates the applicability of the Sections after the initial determinations of seismic performance categories.

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* For derivation see sub-section 2.424 Section 4B

APPENDIX 1 - gives brief guidance for walls other than hollow unit masonry e.g. brick, stone.

TABLE 2.4.1
SECTIONS OF THIS MASONRY CODE TO BE USED FOR DESIGN
OF DIFFERENT CATEGORIES OF BUILDINGS

	Seismic Performance Category A		Seismic Performance Category B			Seismic Performance Category C		
	All Bldgs* under 10.64m	Bldgs* over 10.64m	Single Storey	Other Bldgs* under 10.64m	Bldgs over 10.64	Single Storey	Other Bldgs* under 10.64m	Bldgs over 10.64m
Hollow Units:								
Structural	4C	4A	4C	4A	4B	4C	4B	4B
Non-Structural	4C	4C	4C	4C	4A	4C	4C	4A
Other Masonry				APPENDIX I				

* Note that the Sections named provide the main basis for design in each case but that especially in the case of 4A and 4B some cross referencing is inevitable.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

The first number indicates the Part of the Code, the first digit in the second number indicates the Section in the Part, the second and third digits in the second number indicate the sub-section in the Section, and the third number indicates the Article in the sub-section. These are illustrated as follows:

2	Part 2
2.500	Part 2, Section 5
2.506	Part 2, Section 5, sub-section 6
2.506.3	Part 2, Section 5, sub-section 6, Article 3

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

DEFINITIONS

The following definitions provide the meaning of terms used in this Section 4.

Area, Gross Cross-sectional - The total area face-to face of masonry including cells or cavities of a section perpendicular to the vertical direction of loading.

Area, Net Bedded - The actual area of masonry units that bear on the mortar bed with deductions for rakes and similar joint treatments. In grouted construction the continuous vertical filled grout cores or grout spaces are included.

Area, Net Cross-sectional of Hollow Unit - The gross cross-section area of a section minus the average area of ungrouted cores or cellular and other spaces.

Area, Net Vertical Shear - The minimum gross cross-sectional area of a plane of hollow units less their ungrouted cores or the mortar contact areas at head joints, whichever is less.

Blocks (Hollow) - A block with cavities passing through the block so that the remaining solid material is between 50% and 75% of the total volume of the blocks based on overall dimensions. Blocks for use in load-bearing walls shall have a proportion of solid material between 50% and 60%.

Bond, Running - When in a wythe, at least 75% of the units in any transverse vertical plane lap the ends of the units above and below a distance not less than 40 mm or one-half the height of the units. whichever is greater; the wythe, for the purpose of this document, shall be considered to be laid in running bond. (Note that for the purpose of this definition center bond or half bond is not necessarily required to obtain running bond). Where corners and wall intersections are constructed in a similar fashion, they shall be considered to be laid in running bond.

Bond, Stacked - All conditions of head joints not qualifying as running bond and all continous vertical joints (excepting true joints such as expansion and contraction joints) shall be considered to be stacked bond construction.

Dimensions - Overall dimensions for masonry units and walls are nominal; actual dimensions of unit masonry may not be decreased by more than 40 mm from the nominal dimensions of grout spaces, clearances and cover given are actual.

Effective Eccentricity - The actual eccentricity of the applied vertical load including that caused by member deflections and thermal or other movements of connected members plus the additional eccentricity which would produce a moment equal in magnitude to that produced by lateral loads.

Effective Peak Acceleration (A_a) - A value related to anticipated ground motion

Importance Factor - A factor which depends on the essential nature of the facility for post earthquake recovery, or the likely number or relative mobility of its occupants.

Joint Bed - The horizontal layer or mortar on or in which a masonry unit is laid.

Joint Head - The vertical mortar joint between ends of masonry units.

Joint Shoved - Produced by placing a masonry unit on a mortar bed and then immediately shoving it a fraction to effect solid, tight joints.

Load Bearing - Synonymous with Structural.

Masonry Unit - Block conforming to the requirement specified in this Section.

Non-load Bearing - Synonymous with Non-structural.

Non-structural - This term refers to components or systems which do not serve in providing resistance to loads or forces other than induced by their own weight. Walls that enclose a building or structure's interior are structural components.

Partially Reinforced Masonry - Masonry construction conforming to Section 4C and other applicable provisions of this Section.

Reinforced Masonry - Grouted masonry construction conforming to Section 4B of these requirements.

Reinforcement Ratio - This is the ratio of the areas of reinforcement to the gross cross-sectional area of the masonry perpendicular to the reinforcement.

Seismic Performance Category - Categories ascribed to building by use of the letters A,B,C, and D and derived from the importance factor assigned to the building and the effective Peak Acceleration assigned to the ground on which the building is sited.

Shear Wall - Is a vertical component resisting lateral forces by in-lane shear and flexure. Nominally all load bearing walls in masonry buildings shall also be checked for the effect of induced lateral loads.

Structural - This term refers to a system or component which serves in providing resistance to loads or forces other than induced by the weight of the element itself. All portions of the seismic resisting system are structural, but not all structural components need be part of the seismic resisting system. Bracing components, bracing systems, and all walls that enclose a building or structure's exterior are structural elements.

SYMBOLS AND NOTATIONS

A_a	effective peak acceleration
A_{cc}	area of concrete in compression
A_v	coefficient representing effective peak velocity-related acceleration
b	width or thickness of wall
C_d	deflection amplification factor
C_s	seismic design coefficient
d	depth or length of wall storey distance in feet
e	eccentricity
f_a	applied axial compressive stress
F_a	allowable compressive stress
f_m	applied flexural stress
F_m	allowable flexural stress
f_{am}	combined maximum compressive stress due to axial load and bending
f_m'	ultimate compressive strength value
F_p	seismic force
f_v	permissible tensile stress in steel
F_x	lateral seismic shear force
h	height
I	importance factor
k	coefficient
kg	kilogram
L	overall length of building
$m(m^2, m^3)$	metre (square metre, cubic metre)
ml	millilitre
$mm(mm^2)$	millimetre (square millimetre)
N	Newton
P	load
Q_x	total deflection
R	response modification factor
S	site coefficient

s	spacing of stirrups
T	fundamental period of building
t	thickness of wall
$\emptyset$	capacity reduction factor
θ	stability coefficient
V	shear load
V_x	seismic shear force
v	shear stress
W	total gravity load
Z	zone factor
Δ	design storey drift
Δ_a	allowable storey drift

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 4A
GENERAL REQUIREMENTS FOR BLOCK MASONRY

PART 2
SECTION 4A

2.400 BLOCK MASONRY - GENERAL

2.401 Background to Section 4A

2.401.1 Section 4A is the basic Section around which the provisions for hollow unit masonry are developed.

2.401.2 It is appropriate for blockwork subjected to moderate to heavy working stresses or in areas of low to moderate seismicity, e.g. low buildings in Seismic Performance Category C, or multi-storey construction for buildings in Seismic Performance Category A.

2.401.3 It represents typical construction practices in the region and is largely based on work done by the Jamaica Institution of Engineers in 1974, ratified by the Council of Caribbean Engineering Organisation in 1975, for inclusion in the then proposed Caribbean Uniform Building Code.

2.401.4 Areas not covered in that document were:

(a) Detailed provisions for seismic design. Reference was made to the SEAOC Code* which makes no provision for varying construction quality with seismicity or performance category.

(b) Special reduced provisions for lightly stressed walls (e.g. single-storey masonry buildings), which are widely used in the region for low cost residential construction.

2.401.5 Section 4A provides for general requirements for materials and for designs for gravity loads whereas Sections 4B and 4C deal with items (a) and (b) above respectively.

2.402 Scope

2.402.1 These requirements in Section 4A cover the use of hollow concrete blocks from Portland Cement, water and natural aggregates and used in loadbearing walls. Reference must be made to Section 4B for additional seismic requirements for seismic performance category B-D and buildings over 10.5 metres high in seismic performance Category A.

2.402.2 Sections 4A and 4B may be by-passed completely for buildings or elements falling in the categories assigned to Section 4C as indicated in Table 2.4.1

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* SEAOC Code - Recommended Lateral Force Requirements by Structural Engineers Association of California.

2.403 Block Units

2.403.1 DIMENSIONS AND TOLERANCES - Concrete blocks shall be standard nominal sizes 400 mm long x 200 mm high.

2.403.2 Standard nominal thickness shall be 100 mm, 150 mm or 200 mm. Tolerances shall be ± 3 mm.

2.403.3 Other sizes of blocks may be used provided that they conform with one of the standards listed in this clause.

2.403.4 CAVITIES - Blocks shall be 2 hole pattern and shall have a net cross-sectionsl area not less than 50 percent and not greater than 60 percent of the gross section. Minimum face shell and web thickness shall be 25 mm.

2.403.5 Where open-ended units are used, minimum web thickness shall be 35 mm.

2.403.6 MANUFACTURE AND PHYSICAL REQUIREMENTS - blocks used for construction in accordance with this section shall be hollow concrete blocks. Sizes, manufacture and physical requirements of hollow concrete blocks shall be in accordance with one of the following Standards:

a) JS.35:1975 Jamaican Standard Specification for Standard Hollow Concrete Blocks - Jamaica Bureau of Standards.

b) ASTM C90 - Hollow Load Bearing Concrete Masonry Units.

c) ASTM C140 - Method of Test for Concrete Masonry Units. (American Society for Testing and Materials).

d) BS 2028, 1362:1968 - British Standard for Precast Concrete Blocks.

e) The corresponding local standard for the relevant Caribbean country.

2.403.7 The manufacturer shall state the standard to which blocks are supplied, but in any case, special requirements stated in this section shall take precedence over the above listed standards.

2.403.8 ABSORPTION - Maximum absorption of water shall not exceed 208 kg/m^3 of normal weight concrete for Grade A blocks when tested in accordance with ASTM C140.

2.403.9 DRYING SHRINKAGE - Concrete masonry units used for structural purposes shall have a maximum linear shrinkage of 0.065 percent from the saturated to the oven-dry condition.

2.403.10 COMPRESSION TEST - Test for compression strength of blocks shall be made with cells vetical and shall be made at 28 days in accordance with teh corresponding standard listed at clause 2.403.11. Stress at failure is based on gross area of section of the specimen. For consignments of over 10,000 blocks, a test shall be carried out for every 10,000 blocks or part thereof. The size of sample for each test shall conform with the applicable material standard but shall in no case be less than 10 randomly selected blocks. For consignments of less than 10,000 blocks, a test shall be carried out for each 5,000 blocks or part thereof. In this latter case, each test shall consist of 5 blocks.

2.403.11 Listed below in Table 2.403.1 are some suggested designations and required strengths for hollow load-bearing concrete masonry units.

TABLE 2.403.1
SUGGESTED STRENGTHS FOR HOLLOW
CONCRETE MASONRY UNITS

Block Type	Minimum Compressive Strengths on Gross Cross Sectional Area-N/mm^2	
Designation*	Average of 10 Blocks	Lowest Individual Block
A (14)	14	11
A (10.5)	10.5	8.4
A (7)	7	5.6
B (5)	4.9	4.2

* Grade A - For general use exterior and interior load bearing and non-load-bearing purposes.
Grade B - For general use non-load-bearing purposes only.

2.403.12 MOISTURE CONTENT ON DELIVERY - Moisture content of blocks on delivery to site shall not exceed 35 percent of the maximum absorption specified in this clause.

2.404 Mortar

2.404.1 MIXES AND STRENGTHS - Mortar for block joints shall be proportioned with parts by volume of cement:lime:sand, as follows.

TABLE 2.404.1
MORTAR MIXES

Block Type	Mortar Mix	Minimum Compressive Strength on 50 mm Cubes at 28 Days N/mm^2
A 14	1:0:3	22.4
A 10.5	1:0-1/2:3	16.8
A 7	1:0-1/2:4	11.2
B	1:0-1/2:6	-

In each case, the lime fraction is optional. Where ordinary cement with a plasticiser is used, the lime fraction of the mix shall be omitted. Mortar shall otherwise be in accordance with ASTM476.

2.404.2 MATERIAL STANDARD FOR MORTAR AND IN-FILL - Material standards for cement, lime and fine aggregate for mortar and concrete in-fill shall be from the following clauses as applicable.

2.405 Concrete In-fill Cavities

2.405.1 NORMAL CONCRETE IN-FILL - Concrete for filling block cavities shall be a nominal mix of parts by volume of cement: fine aggregate: coarse aggregate or of compressive strength as shown in Table 2.405.1.

TABLE 2.405.1
PROPERTIES OF CONCRETE IN-FILL

Block Type	Minimum Compressive Strength on 150 mm Cubes at 28 Days N/mm^2	Concrete Mix for Cavity In-fill
A (7)	15.8	1:3:6*
A (10.5)	21	1:2:4*
A (14)	21	1:2:4*

2.405.2 Maximum aggregate size shall be 19 mm but in no case greater than one-quarter times the minimum dimension of the cavity.

2.405.3 Water cement ratio may exceed conventional ratios for normal concrete provided that slumps specified in clause 2.405.9 are not exceeded.

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* Where in-fill height exceeds 600 mm, coarse grout shall be used.

2.405.4 Material standards shall be as listed in this sub-section. Alternative in-fill concrete may be proportioned in accordance with the rules in Part 2 Section 6 for normal reinforced concrete provided that the required strengths are achieved.

2.405.5 COARSE GROUT - When aggregates are selected in accordance with ASTM C404 the following shall govern:

2.405.6 For in-fill coarse grout, to be used where, the filled cavity is not less than 76 mm in width and 70 sq. cm. in area.

2.405.7 For grout space between multi-wythe construction, the minimum space must be 50 mm between wythes.

2.405.8 Coarse grout in-fill shall be composed by volume of one part, to which may be added not more than 1/10 part hydrated lime, two to three parts sand and one to two parts gravel. Larger proportions of gravel may be used in larger grout spaces.

2.405.9 Grout shall have a consistency to fill all spaces considering the method of consolidation but slumps shall be between 115 mm and 230 mm.

2.406 Reinforcing Steel

2.406.1 Reinforcing steel shall be steel to any of the following standards.

American Standard for Testing and Materials	- ASTM A615
American Standard for Testing and Materials	- ASTM A616
American Standard for Testing and Materials	- ASTM A617
American Standard for Testing and Materials	- ASTM A706
British Standards Institution	- BS 4449
Jamaica Bureau of Standards	- JS:33/1974

2.407 Water

2.407.1 Water for mixing concrete or mortar shall be from the mains and otherwise shall be clean and free from injurious amounts of oil, acid, alkali, organic matter or other harmful substances.

2.408 Storage and Protection

2.408.1 STACKING OF BLOCKS - Blocks shall be stacked and stored in a dry place which is not in contact with dirt.

2.408.2 Exposed stacks shall be covered from rain by protective material to prevent increase in moisture content.

2.408.3 STORAGE OF OTHER MATERIALS - Cement and lime shall be stored at least 150 mm above ground in a dry place and under cover at all times. Aggregate shall be piled on a clean, hard surface.

2.409 Construction

2.409.1 LAYING AND BOND - Blocks shall be laid with mortar across all webs and face shells and in vertical joints. Blocks shall generally be laid in stretcher or running bond.

2.409.2 In special cases where stack bond is used in load-bearing walls special horizontal reinforcement is required as defined in clause 2.416.9. All defined load-bearing walls shall intersect and end on poured reinforced concrete stiffeners except as indicated below. Stiffener columns at the intersection and ends of load-bearing walls may be omitted for the conditions set out in sub-section 2.413.

2.409.3 Inclusive of the provision at ends and intersections, wall panels shall be provided with concrete stiffener columns and belt beams as specified in sub-sections 2.413 and 2.414 Section 4A of these recommendations with exceptions as stated.

2.409.4 Half-blocks shall be provided at junctions with stiffeners to complete bond courses.

2.409.5 Alternatively, blocks shall be cut to a neat vertical line such as that made by a mechanical saw. Where blocks are cut, at least two webs shall be left intact.

2.409.6 RATE OF CONSTRUCTION - Units shall be placed while mortar is still soft and plastic. Mortar shall be discarded if not used before lapse of the following times.

Where cement:lime ratio is 2.0 or less - 2 hrs.
Where cement:lime ratio is greater than 2.0 - 1 hr.

2.409.7 The minimum period which shall elapse before loads other than own-weight loads are applied to a newly constructed blockwall shall be determined from the design loading conditions. This shall in no case be less than four days.

2.409.8 INSERTION OF REINFORCING BARS - Blocks shall be strung over verical reinforcing bars in place, or bars shall be inserted into the cavities of blocks ensuring adequate lap. Alternatively, where precast open-ended blocks are used or where end webs are provided with "knock-out" recesses, head joints shall be fully grouted.

2.409.9 SEALING UNFILLED CAVITIES - Precautions shall be taken to seal specified unfilled cavities to prevent loss of concrete from belt beams during casting.

2.409.10 CASTING STIFFENER COLUMNS AND BELT BEAMS - Stiffener columns and belt beams shall be cast after adjacent blockwalls have been built up to beam level. Stiffener columns shall be poured and vibrated in lifts not exceeding 1.8 m. If necessary, openings shall be provided in full height forms to facilitate this.

2.409.11 The concrete in belt beams shall be placed to bond to the masonry units and shall not be separated there from by wood, felt or any other material which may prevent bond.

2.409.12 REINFORCEMENT - Reinforcement shall be established in accordance with Section 4B - Design. Laps and anchorage to bars shall be in accordance with sub-section 2.412 Section 4A.

2.409.13 Bars shall be continous through, or fully anchored in belt beams and stiffener columns where they intersect with these members. Bars shall be free of rust, grease, dirt, hardened grout and other deleterious substances prior to casting.

2.409.14 CASTING OF FOUNDATION CONCRETE - All walls shall be provided with foundations and starter bars shall be embedded therein. Size and depth of foundations shall be as designed to restrict ground pressures to safe values and to restrict settlement to acceptable limits.

2.409.15 Top of foundations shall be kept clean and free of loose concrete and deleterious substances prior to commencement of erection of blockwalls.

2.409.16 Top of foundation concrete shall be finished with at least 3 mm amplitude to provide bond with first course.

2.409.17 CONCRETING CAVITIES - Concreting of block cavities shall proceed at intervals not exceeding 600 mm height, or three courses, whichever is greater. Concrete poured in these cavities shall be thoroughly rammed and consolidated using a 16 mm diameter rod or similar device. Each lift shall be terminated at least 12 mm below the top of the block to ensure a lateral key. Where "coarse grout" as defined by sub-section 2.404 is used "low-lift" grouting to a maximum height of 1.3 m is acceptable. "High-lift" grouting exceeding the above heights may be used in accordance with ATC-3*, provided that concrete mix proportions and aggregate size are varied accordingly to the approval of the Regulatory Authority.

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* Applied Technology Council ATC-3.06 - Seismic Regulations for Buildings.

2.409.18 MORTAR JOINTS - Mortar joint thickness shall be 10 mm. Tolerance shall be 3 mm subject to a minimum thickness equal to the diameter of any horizontal reinforcement.

2.409.19 MINIMUM LINTEL SEATING - Lintel seating in blockwalls shall be a minimum of 200 mm.

2.409.20 REINFORCEMENT AROUND OPENINGS - Where no reinforced concrete stiffeners are adjacent to openings, block cavities adjoining these openings shall be provided with a vertical bar of 12 mm minimum diameter and the cavities shall be filled with concrete.

2.409.21 A minimum of 12 mm diameter horizontal bar or equivalent area of smaller bars shall be provided in the mortar joint adjacent to the sill for all openings and shall extend not less than 600 mm beyond the edge of the opening. 12 mm diameter bar shall be permitted where bond beams are used or where the bed surface of webs are provided with an appropriate recess.

2.409.22 TEMPORARY BRACING TO HIGH WALLS - Under high wind conditions, concrete block walls shall be adequately braced against overturning.

2.409.23 Under normal conditions up to wind speeds of say 50 kilometres per hour, the unbraced heights of walls exposed to wind pressure during construction shall not exceed the following:

For 200 mm walls - 4 m
For 150 mm walls - 2.5 m
For 100 mm walls - 1 m

2.409.24 EMBEDDED PIPES AND CONDUIT - For the quality of blockwalls covered by these recommendations, no horizontal or diagonal chases for pipes and conduits shall be permitted without the approval of the Engineer.

2.409.25 Any vertical pipes or conduits to be inserted, shall preferably be installed within the wall during construction, thus avoiding any cutting or damage to wall material.

2.409.26 In unavoidable circumstances, vertical chases may be cut for pipes or conduits not exceeding 12 mm in diameter, provided that:

a) The maximum dimension of the chase does not exceed 40 mm.

b) Minimum concrete cover to pipes or conduit is 12 mm.

c) Where a chase is cut into the face shell of an unfilled cavity, the pipe or conduit shall be inserted into the cavity and the cavity filled with concrete.

2.410 **Field Testing**

2.410.1 FIELD TEST FOR CONCRETE IN-FILL TO CAVITIES - Tests for compressive strength of concrete in-fill shall be carried out as follows:

2.410.2 Each test sample shall consist of three 150 mm cubes taken from a representative batch of the in-fill concrete. Sampling and testing of these cubes shall be in accordance with BS 1881. At least one test shall be made for each 450 sq. m. or wall or part thereof or for each storey height.

2.410.3 For each test, the average compressive strength of the three cubes shall not be less than the average compressive strengths specified in sub-section 2.405 Section 4A, of these recommendations.

2.410.4 In addition, the lowest individual strength result shall not be more than 20% below the average value.

2.410.5 FIELD TEST FOR MORTAR - Tests for compressive strength of mortar shall be carried out as follows:

2.410.6 Each test sample shall consist of three 50 mm cubes taken from a representative batch of the mortar.

2.410.7 Sampling and testing of the mortar shall be in accordance with Tentative Specifications for Mortar for Unit Masonry ASTM C270. At least one test shall be made for each 450 sq. m of wall or part thereof or for each storey height.

2.410.8 For each test, the average compressive strength of the three cubes shall not be less than the minimum compressive strengths specified in sub-section 2.404 Section 4A of these recommendations. In addition, the lowest individual strength result shall not be more than 20% below the average value.

2.410.9 PRISM TEST

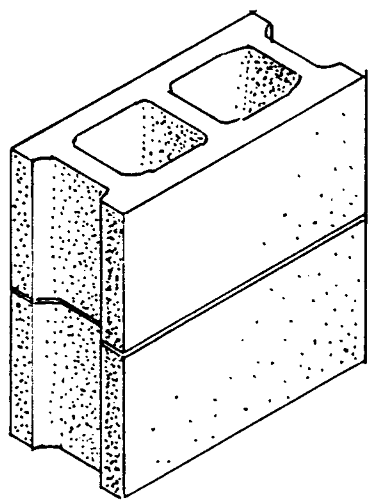
a) Number of Specimens:

Testing shall include tests in advance of beginning operations and at least one field test during construction for each 450 sq. m of wall area. Each prism test shall consist of at least three specimens.

b) Size of Specimen:

The thickness of a specimen shall be the same as the thickness of the wall in the structure. The length of a specimen shall be equal to or greater than the thickness. The height of a speciment shall be at least twice the thickness of the specimen and a minimum of 380 mm. (See Fig. 2.410.1). Prisms shall preferably be 3 units high with two mortar joints but shall in any case be a minimum of two units high.

FIGURE 2.410.1
PRISM TEST SPECIMEN



c) Construction of Specimen:

Each test specimen shall be built by the masonry contractor using the materials and workmanship to be used in the structure. The workmanship to be used in the structure for the mortar bedding, the thickness and tooling of joints, the grouting, the condition of the units and the bonding arrangement shall be included in the specimen.

Where walls are to be single wythe using units 250 mm or more in length, the specimen shall be built in stacked bond.

Where specimens are made during construction, they shall be built at the job site by the masons in a place where they will not be disturbed for 48 hours and subjected to air conditions similar to that in the masonry structure.

d) Handling and Curing Conditions:

After 48 hours, each specimen built at the job site shall have a 20 mm plywood board the same size as the specimen placed top and bottom and tightly wrapped with 2 wire ties to keep the joints from being distrubed and then be transported under the direction of the testing laboratory to the curing room.

e) Capping and Testing:

Each specimen shall be capped and tested in compression in accordance with the applicable procedure set forth in ASTM C140.

f) Calculating Test Results:

The value of f_m' shall be the average of all specimens tested but shall not be more than 125 percent of the value of the lowest individual test specimen. The value of f_m' shall be computed by dividing the ultimate load by the net cross-sectional area of the specimen. The net area in this case shall include concrete in-fill portions, only where the specimen is grouted to represent cases where all cavities in the finished wall are to be solidly grouted.

Prism correction factors for height of prism shall be as follows:

Ratio of h/d	1.5	2.0	3.0	4.0	5.0
Correction Factor	0.86	1.00	1.20	1.30	1.37

2.411 **Loads**

2.411.1 Structures shall be designed for the most severe combination of dead, live and wind or earthquake loads.

a) Dead and Live Loads:

Dead and live loads shall be determined in accordance with Part 2 Section 1 of this Code.

b) Wind Loads:

Wind loads shall be determined in accordance with: Part 2 Section 2 of this Code.

c) Earthquake Loads:

Earthquake loads shall be determined in accordance with: Section 4B sub-sections 2.431 through 2.438 of this Code.

2.412 **Minimum Wall Thicknesses and Reinforcement**

2.412.1 Walls shall be designed to meet the stress restrictions set out hereafter but in any case shall meet the following additional requirements.

2.412.2 MINIMUM WALL THICKNESS - Exterior walls and fire walls shall be a minimum of 150 mm nominal thickness.

2.412.3 Interior non-load bearing walls shall be a minimum of 100 mm nominal thickness subject to requirements for grouted and reinforced joints and cavities.

2.412.4 The ratio of height between effective restraints to thickness shall not exceed 25 for any wall.

2.412.5 For the special case of single storey warehouse walls, these values may be exceeded where the compression stress from vertical design loads does not exceed 0.04 f_m' and where ultimate design analysis, utilising properly substantiated slender-wall design procedures are submitted and approved by the Regulatory Agency.

2.412.6 MINIMUM REINFORCEMENT - The minimum area of reinforcement in both the horizontal and vertical directions shall be 0.07 percent of the gross cross-sectional area of the wall, and the sum of gross in both directions shall be at least 0.2 percent. The maximum spacing for reinforcing bars shall be 1.2 m. Minimum bar size shall be 10 mm.

2.413 Provisions for Stiffener Columns

2.413.1 SPACING OF STIFFENER COLUMNS - Spacing of stiffener columns shall be required in walls of unit masonry as follows:

a) The maximum area of wall panels of 150 mm or 200 mm thick unit masonry, as measured between the concrete members which frame the panels, such as the beams and stiffener columns shall not exceed 23 sq. m.

b) At intervals not exceeding 6 m between columns.

c) At corners and junctions of load-bearing walls.

d) At the end of load-bearing walls.

2.413.2 The foregoing requirements (a) and (d) above may be omitted for the following conditions:

a) Walls for single storey buildings with light roofs in all seismic performance categories. (See Section 4C for provisions for such buildings).

B) Single and two-family residential buildings of two storeys designed and constructed in accordance with all other requirements of this section and provided with vertical reinforcing, links and concrete in-fill to cavities at all intersections as described for single-storey units in Section 4C.

c) All buildings in seismic performance Category A.

2.413.3 Concrete stiffener columns shall also be required adjacent to any corner opening or any opening if the omission of the columns would result in stresses in the blockwork greater than that permitted under sub-section 2.416.

2.413.4 Structurally designed columns may be substituted for the stiffener columns herein required.

2.413.5 In any section of a masonry wall where openings are arranged, to leave a load bearing section of wall less than 400 mm wide, such section shall be of steel or reinforced concrete.

2.413.6 SIZE OF STIFFENER COLUMNS - Stiffener columns shall be not less than 300 mm in width and of a minimum thickness not less than that of the wall.

2.413.7 REINFORCEMENT OF STIFFENER COLUMNS - Stiffener columns shall be reinforced with a minimum of 4 No bars of minimum 12 mm diameter mild steel or of area not less than 0.010 times the gross cross-sectional area of the concrete, tied with 6 mm minimum diameter links spaced not more than 3 mm apart.

2.413.8 The spacing of links less than 450 mm distant from or 1/6 clear space between beams intersecting with the columns, shall be reduced to 150 mm.

2.413.9 Ends of stirrup bars shall bend 135° around longitudinal bars and extend 100 mm beyond.

2.413.10 Vertical reinforcement shall be anchored to the footing and splices shall be lapped 30 bar diameters, for deformed bars and 40 diameters for plain bars.

2.413.11 The cover to the vertical reinforcement shall be 25 mm minimum.

2.414 Provisions for Belt Beams

2.414.1 SPACING OF BELT BEAMS - Belt beams of reinforced concrete shall be placed in all walls of unit masonry, at each floor or roof level and at such intermediate levels as may be required to limit the vertical height of the masonry panel to 5 m.

2.414.2 Bond beams may be used in lieu of poured concrete belt beams.

2.414.3 SIZE AND REINFORCEMENT OF BELT BEAMS - A belt beam shall be not less in dimension or reinforcement than required for the conditions of loading nor less than the following minimums.

2.414.4 SIZE AND REINFORCEMENT OF BELT BEAMS - The width of a belt beam shall be not less than the thickness of the wall supporting it. The depth of such a beam shall be not less than 200 mm. It shall be reinforced with not less than 4 No 12 mm diameter mild steel reinforcing bars placed two at the top and two at the bottom of the beam, or as otherwise required for the condition of loading.

2.414.5 Stirrups shall be not less than 6 mm diameter mild steel bars at 300 mm centres or the depth of the beam whichever is less. Stirrup bars shall have 135^{0} bend and 100 mm extension.

2.414.6 Wall bars shall be continous through or be fully anchored within the belt beam.

2.414.7 CONTINUITY OF BELT BEAMS - The belt beam shall be continous. Continuity of the reinforcement in straight runs shall be provided by lapped splices not less than 30 diameters long for deformed bars or 40 diameters for plain mild steel bars. Continuity at columns shall be ensured at wall ends and corners by providing positive anchorage to the main reinforcement.

2.414.8 Continuity at columns shall be provided by horizontal reinforcement into the columns a distance of 30 diameters. Any change in level of a belt beam shall be made at a column. A belt beam may follow the rake of a gable or shed end.

2.415 Ultimate Compression Strength

2.415.1 VALUES OF f_m' - The value of f_m' shall be either as established in accordance with the Prism Test (clause 2.410.9 Section 4A) or as set out in Table 2.415.1 hereunder.

a) Walls with filled cavities where no prism test made.

Table 2.415.1 expresses recommended values of f_m' based on the least compressive strength of block units, in-fill concrete and mortar proposed in sub-section 2.403, 2.404 and 2.405 respectively.

For this consideration recommended crushing strengths of blocks from sub-section 2.403 are adjusted from gross to net values by use of a factor 1.67.

f_m' is taken equal to 0.7 times the least of the three strength values mentioned above.

TABLE 2.415.1
ULTIMATE COMPRESSIVE STRENGTHS VALUES OF f_m'

Minimum Compressive Strength of Block Units Based on <u>Gross</u> Cross-sectional Area	Assumed Compressive Strength f_m' on Net Area of Masonry N/mm^2
7.0 N/mm^2 (A7)	7.0
10.5 N/mm^2 (A10.5)	10.5
14.0 N/mm^2 (A14)	14.0

b) Walls with some cavities unfilled or where adequate supervision is not available.

For walls with some cavities unfilled, assumed net compressive strength of masonry f_m' shall be taken to be the lesser of

0.7 x mortar strength
or
0.7 x the net compressive strength of the block units.

Where cavities are filled, but adequate supervision is not available, walls shall be designed as if cavities were unfilled.

2.416 **Allowable Stresses**

2.416.1 MASONRY STRESSES - GENERAL - The allowable stresses shall not exceed the weakest of the combination of units and mortars, unless f_m' is determined by tests.

TABLE 2.416.1
ALLOWABLE STRESSES IN REINFORCED MASONRY

Type of Stress	Where "Special Inspection Procedures Required" as Appendix 2: Related to f_m'	Where "Special Inspection Procedures Required" as Appendix 2: Maximum	"Special Inspection Procedures" not required
COMPRESSIVE AXIAL	See Clause 2.416.3		One-half of the values permitted under Cl. 2.416.3
FLEXURAL	$0.33\ f_m'$	7.0 N/mm^2	$0.166\ f_m'$ but not exceed 3.5 N/mm^2
SHEAR*			
Masonry takes shear:			
Flexural Members	$1.1\sqrt{f_m'}$	0.35 N/mm^2	0.17 N/mm^2
Shear Walls $M/Vd \geqslant 1$	$0.9\sqrt{f_m'}$	0.28 N/mm^2	0.14 N/mm^2
$M/Vd = 0.25$*	$2\sqrt{f_m'}$	0.35 N/mm^2	0.17 N/mm^2
Reinforcement taking entire shear:			
Flexural Members	$3\sqrt{f_m'}$	1.06 N/mm^2	0.35 N/mm^2
Shear Walls $M/Vd \geqslant 1$	$1.25\sqrt{f_m'}$	0.53 N/mm^2	0.24 N/mm^2
$M/Vd = 0.25$*	$2\sqrt{f_m'}$	0.84 N/mm^2	0.42 N/mm^2

TABLE 2.416.1 (continued)

BEARING	0.25 f_m'	3 N/mm^2	0.125 f_m' but not to exceed 2.8 N/mm^2
MODULUS OF ELASTICITY	600 f_m'	14000 N/mm^2	500 f_m' but not to exceed 10,500 N/mm^2
MODULUS OF RIGIDITY	240 f_m'	5600 N/mm^2	200 f_m' but not to exceed 4200 N/mm^2

* The allowable stress shall be the stress related to f_m' but not to exceed the maximum stress listed here.

Interpolate by straight line for M/Vd values between 0.25 and 1.

M is the Maximum bending moment occurring simultaneously with the shear load V at the section under consideration.

2.416.3 FORMULA FOR ALLOWABLE AXIAL STRESSES - The allowable compressive stress for axial loads based on the net horizontal cross section of the wall shall be:

$$F_a = 0.20 f_m' \left[1 - \left(\frac{h}{40t} \right)^3 \right] \qquad \text{Eqn 2.416.3.1}$$

Where:

h = Effective or unsupported height of the wall and

t = Nominal thickness of wall

f_m' = Masonry compressive strength (on net area) as determined by sub-section 2.415. The value of f_m' shall not exceed 21 N/mm^2.

Where "Special Inspection Procedures" as Appendix 2 are not required, f_m' shall be taken as half the value obtained from Eqn. 2.416.3.1

2.416.4 COMBINED STRESSES - Where members are subject to combined axial and flexural stresses, the condition shall be dealt with as follows:

a) Where the eccentricity e is less than $\frac{t}{10}$, the effects of eccentricity may be ignored

b) where e is less than or equal to $\frac{t}{6}$,

$\frac{f_a}{F_a} + \frac{f_m}{F_m}$ shall not exceed 1

f_a = Applied axial compressive stress

F_a = Allowable compressive stress

f_m = Applied flexural stress

F_m = Allowable flexural stress

c) Where e, is between $\frac{t}{6}$ and $\frac{t}{3}$

$$0.5\, f_{am}\left[\frac{1}{F_a} + \frac{1}{F_m}\right] \leqslant 1$$

Where f_{am} = the combined maximum compressive stress due to axial load and bending.

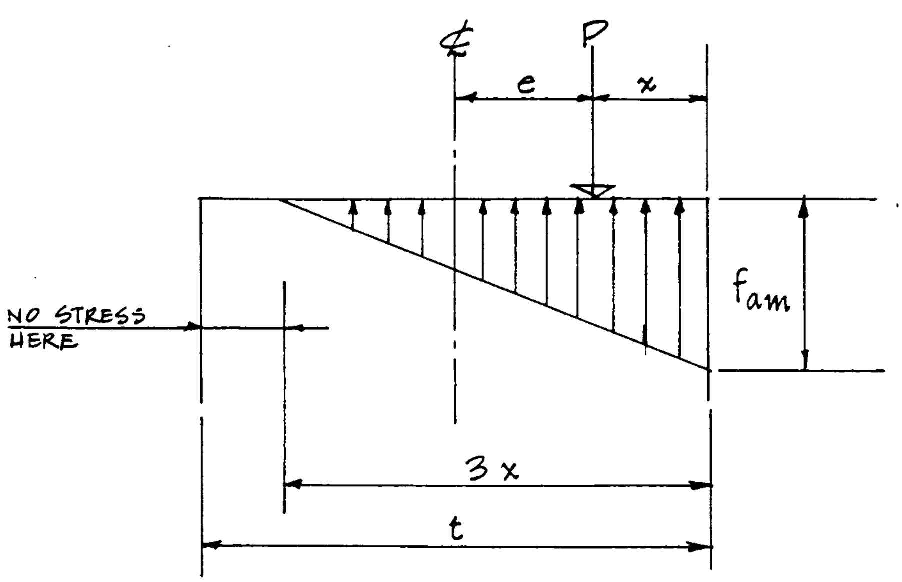
f_{am} shall be calculated using the assumption that no tension is permitted in the material and using the reduced bearing surface resulting.

f_{am} shall be calculated as follows:

If P = load/mm run of wall and acts as indicated in Figure 2.416.1

$$f_{am} = \frac{2P}{3x} \text{ or } \frac{2P}{3\left(\frac{t}{2} - e\right)}$$

FIGURE 2.416.1
TYPICAL LOADING FOR AXIAL AND FLEXURAL STRESSES



d) Where the vertical eccentricity exceeds t/3, or a value which would produce tension in the reinforcement, the allowable load shall be determined on the basis of a transformed section and linear stress distribution. Reinforcement in compression shall be neglected. The compressive stress in the wall shall not exceed the flexural compressive stress f_m.

Consideration shall be given to placing tension steel near to the tension face of the wall.

e) In the case where the virtual eccentricity exceeds t/3 or a value which would produce tension in the reinforcement, but where the actual compression stress f_a, does not exceed 0.1 x the allowable axial stress F_a or 0.1 flexural compressive stress f_m, the section may be designed for bending only, ignoring the effect of axial compression.

For this case, tension steel shall be positioned as close to the tension face of the wall as possible.

2.416.5 REINFORCEMENT STRESSES - The stresses in steel reinforcement used in masonry where bending predominates as defined in clause 2.416.4 (d) and (e) should not exceed the following limits:

(a) Mild steel (or ASTM Grade 40) Bars - 140 N/mm^2
(b) High Yield Steel (ASTM Grade 60) - 168 N/mm^2

2.416.6 STRESSES DUE TO WIND AND EARTHQUAKE FORCES - Wall panels shall be designed to resist all forces due to the effects of wind and earthquake. For stresses due to wind or earthquake forces combined with dead and live loads, the allowable stresses may be increased by 33 1/3 percent, provided that the strength resulting is not less than that required for combined dead and live loads.

2.416.7 An exception is in the case of High Yield steel to BS4449 which should not be stressed above 252 N/mm^2

2.416.8 Where earthquake or wind forces are expressed as ultimate loads for design, strength design methods shall be used, assuming strength reduction factors as set out in sub-section 2.430 and load factors as set out in sub-section 2.438.

2.416.9 EFFECTS OF STIFFENER COLUMNS ON WALL STRENGTH - Stiffener columns shall be provided as specified in sub-section 2.413. The presence of poured concrete stiffener columns shall be dealt with as follows:

(a) Where the spacing of stiffener columns is less than or equal to 24 t, the contribution of the masonry wall and the stiffener columns shall be calculated separately using the expression,

Safe Load = $F_a \times A_{cc}$

Where F_a = Allowable axial compressive stress from Table 2.416.1

A_{cc} = Area of concrete in compression.

For concrete in the stiffener columns, f_m' shall be taken to be 0.7 x 28-day cube strength.

(b) Where the spacing of stiffener columns is greater than 24t, the strength of the wall panel shall be based on the assumption that the wall is constructed of masonry only.

2.417 Shear Bond and Anchorage

2.417.1 SHEAR STRESS - The nominal shear stress as a measure of diagonal tension in reinforced concrete masonry members shall be computed by:

$$v = \frac{V}{bd}$$

Where V = shear force
b = width or thickness of wall
d = depth or length of wall.

2.417.2 For design, the maximum shear shall be considered as that at the section a distance 'd' from the face of the support, except in the case of short cantilevers and brackets which shall be given special consideration.

2.417.3 Where applicable, effects of torsion shall be added and effects of inclined flexural compression in variable depth members shall be included. For beam with an I or T section, b' shall be be substituted for b.

b is flange width
b' is web thickness

2.417.4 REQUIRED WEB REINFORCEMENT - Web reinforcement shall be provided to carry the entire shear in any of the following conditions:

(a) Where the computed shearing stress exceeds that allowed for masonry with no shear reinforcement.

(b) Where the compression member is subjected to reversal of stresses.

(c) For the case of masonry "bond" beams, in the portion of the length of a member where negative reinforcement is required and for minimum distance of 1/16 clear span beyond inflection point.

(d) In flexural members, where longitudinal reinforcement is designed to act in compression.

2.417.5 TYPES OF WEB REINFORCEMENT - Web reinforcement may consist of:

(a) Bars or stirrups perpendicular to the longitudinal reinforcement.

(b) Bars or stirrups welded or otherwise rigidly attached to the longitudinal reinforcement and making an angle of 45^{o} or more thereto.

(c) Combination of (a) and (b).

2.417.6 STIRRUPS DESIGN - The area of steel required in stirrups placed perpendicular to the longitudinal reinforcement, shall be computed by:

$Av = Vs/f_{v}d$

f_{v} = permissible tensile stress in steel,

s = spacing of stirrups.

2.417.7 SPACING OF WEB REINFORCEMENT - Where web reinforcement is required, it shall be spaced so that every 45^{o} line (representing a potential crack) extending towards the reaction from mid-depth (d/2) of the member to the longitudinal tension bars should be crossed by at least one line of web reinforcement.

Where web reinforcement is required, its area should not be less than 0.10 percent of the area 'bs' computed as the product of the width of the wall and the spacing of the web reinforcement along the longitudinal axis of the member.

2.417.8 STRESS RESTRICTIONS - Web reinforcement stresses, bond stress and masonry shear stress, shall not exceed those set forth in Table 2.416.1

2.417.9 Shear stresses shall be zero at a control joint and at continuous vertical joints in stacked bond unless specific elements are designed to transmit shear across the joint.

2.417.10 COMPUTATION OF BOND STRESS - In flexural member in which the tension reinforcement is parallel to the compression face, the flexural bond stress at any cross-section should be computed by:

$$u = \frac{V}{\sum ojd}$$

where $\sum o$ = Sum of perimeters of tensile reinforcing bars.

2.418 Lateral Stability

2.418.1 Where load-bearing block walls are used as a lateral load-resisting system to resist wind and earthquake loads, care shall be taken to provide adequate resistance along both major axes of the building.

2.418.2 For example, in the case of cross wall type buildings where there may be inadequate provision of longitudinal walls, some other lateral load-bearing system shall be provided. This might take the form of columns and floor beams behaving as a reinforced concrete frame in this direction.

2.418.3 Full consideration shall also be given to the need for stability of all buildings against overturning and the increased masonry stresses that will result from such overturning.

2.419 Shear Walls

2.419.1 Shear walls shall be those walls selected to resist all lateral forces on the building. Such walls shall be designed and reinforced in accordance with all the foregoing Section 4A.

2.419.2 Other walls, not so selected, may be designed for dead and gravity live loads only, and shall be provided with minimum reinforcement appropriate to "Non-Structural" walls from the appropriate Section of this Code as indicated in Table 2.4.1 in "Guide to Use of Section 4".

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 4B
SPECIAL PROVISIONS FOR SEISMIC DESIGN OF HOLLOW UNIT MASONRY BUILDINGS (OTHER THAN SINGLE STOREY MASONRY BUILDINGS WITH LIGHT ROOFS)

PART 2
SECTION 4B

2.420 Background to Section 4B

The masonry design and construction procedures given in this Code are considered necessary for providing the performance levels implicit in the selection of factors used for seismic forces in these provisions.

Single storey buildings with light roofs and certain other categories of buildings listed in Table 2.4.1 in "Guide to Use of Section 4" have been excepted from these provisions and are dealt with in Section 4C.

The quality and testing of masonry and steel materials and the design and construction of masonry and reinforced masonry components which resist seismic forces shall conform to the requirements of the preceding Section 4A and the references listed therin except as modified by the provisions of this Section.

This Section 4B defines the various seismic performance categories of buildings and presents the special provisions to be made in design and construction for each category over and above the provisions stated in Section 4A.

2.421 Seismic Performance

2.421.1 For the purpose of these provisions all buildings shall be assigned to a Seismic Performance Category.

2.421.2 Any method of analysis or type of construction required for a higher Seismic Performance Category may be used for a lower Seismic Performance Category.

2.421.3 For the purpose of this Caribbean Uniform Building Code, Seismic Performance Category of any building shall be determined by first considering,

(a) the importance factor, which depends on the essential nature of the facility for post earthquake recovery, or the likely number or relative mobility of its occupants

(b) its effective peak acceleration, A_a

2.422 Importance factors for Buildings, I

2.422.1 Factors used shall be 1.5, 1.2 or 1.0. Examples for the three classes of buildings are listed below in Table 2.422.1.

TABLE 2.422.1
IMPORTANCE FACTORS FOR CLASSES OF BUILDINGS

I = 1.5	I = 1.2	I = 1.0
Fire suppression facilities.	Public assembly for 100 or more persons.	All buildings not classified herein as I = 1.5 or I = 1.2
Police facilities.	Open-air stands for 2,000 or more persons.	
Structure housing medical facilities having surgery and emergency treatment areas.	Day-care centres.	
	Colleges.	
Emergency preparedness centres.	Retail stores with 465 m^2 floor area per floor or more than 10.7 m in height.	
Power stations or other utilities required as emergency back-up facilities.	Shopping centres with covered malls, over 2,800 m^2 per floor.	
Communication facilities.	Hotels over 4 storeys in height.	
	Apartment houses over 4 storeys in height.	
	Emergency vehicle garages.	
	Ambulatory health facilities.	
	Hospital facilities other than those assigned I = 1.5.	
	Wholesale stores over 4 storeys in height.	
	Printing plants over 4 storeys in height.	
	Hazardous occupancies consisting of flammable or toxic liquids including storage facilities for same.	

I = 1.5 is assigned to buildings having essential facilities which are necessary for post-earthquake recovery. Such buildings shall be designed with accesses protected from adjacent structures or falling hazards during and after earthquakes.

I = 1.2 is assigned to buildings having a large number of occupants or buildings in which the occupants movements are restricted or their mobility impaired.

2.423 Effective Peak Acceleration, A_a

2.423.1 The following values have been proposed for A_a. Value A_a is related to the anticipated ground motion.

TABLE 2.423.1
EFFECTIVE PEAK ACCELERATION A_a FOR THE CARIBBEAN TERRITORIES

Territory	A_a
Jamaica	0.3
Antigua	0.3
St. Kitts, Nevis	0.3
Montserrat	0.3
Dominica	0.3
North West Trinidad	0.3
St. Lucia	0.3
St. Vincent	0.2
Grenada	0.2
Rest of Trinidad and Tobago	0.2
Barbados	0.15
Guyana	0.1
Belize	N.A.

2.424 Seismic Performance Categories

2.424.1 Seismic performance categories can be assiged to buildings in the various Caribbean Territories by use of Table 2.424.1 below.

2.424.2 These categories are related to the importance factor of the building and the effective peak acceleration assigned to the territory and may be either Category A,B,C, or D.

TABLE 2.424.1
SEISMIC PERFORMANCE CATEGORIES

A_a, Effective Peak Acceleration	I, Importance Factor I = 1.5	I = 1.2	I = 1.0
0.3	D	C	C
0.2	D	C	C
0.15	C	C	B
0.1	B	B	B
0.05	A	A	A

2.424.3 As an approximate conversion from the zone factor format of Part 2 Section 3 to A_a values for use in Table 2.424.2, the following values can be used:

TABLE 2.424.2
APPROXIMATE EQUIVALENTS FOR ZONE FACTOR AND A_a

Z Factor	Approximate A_a Equivalent
0.25	0.1
0.375	0.15
0.5	0.2
0.75	0.3
1.0	0.4

The following describes the requirements of the various seismic performance categories A to D.

2.425 Construction and Material Limitations for Seismic Performance Category A

2.425.1 Buildings assigned to Category A may be of any construction permitted in Section 4A for general load bearing masonry construction.

Exception: The reinforcement provisions of Section 4A need not apply to walls falling in the category designated in Section 4C; that is, walls for single storey buildings with light roofs.

2.426 Construction and Material Limitations for Seismic Performance Category B

2.426.1 Buildings assigned to Category B shall conform to all requirements for Category A and to the additional requirements and limitations of this Section.

Exception: The reinforcement provisions of Section 4A need not apply to walls falling in the category designated in Section 4C; that is, walls for single storey buildings with light roofs.

2.426.2 CONSTRUCTION LIMITATIONS CATEGORY B - Masonry components shall be constructed to conform to the limitations of this Section.

(a) Height Limitations:

Components of the seismic resisting system in buildings under 10.7 m in height shall be reinforced in accordance with Section 4A of this document.

Components of the seismic resisting system in buildings over 10.7 m in height shall be reinforced masonry in accordance with Section 4A, along with the additional requirements for Category B set out in this Section 4B.

(b) Ties:

In addition to the requirements of Section 4A, additional ties shall be provided around anchor bolts which are set in the top of a column or pilaster. Such ties shall be located within the top 100 mm of the member and shall consist of not less than two 12 mm dia or three 10 mm.

(c) Shear Walls:

Shear walls shall conform to the general requirements of load bearing walls in Section 4A.

(d) Non-structural Components:

Non-structural walls, partitions, components shall be designed to support themselves and to resist seismic forces induced by their own weight. Holes and openings shall be suitably stiffened and strengthened.

Non-structural walls and partitions shall be in accordance with the requirements of Section 4C.

2.426.3 Material Limitations Category B - The following materials shall not be used unreinforced for any structural masonry in Category B, apart from exceptions described in Section 4C.

Unburned clay masonry (e.g. oil stabilised clay)

Structural Clay Load-bearing Tile

2.427 Construction and Material Limitations for Seismic Performance Category C

2.427.1 Buildings assigned to Category C shall conform to all the requirements for Category B and to the additional requirements and limitations of this Section.

Exception: See Section 4C for requirements for single storey buildings.

2.427.2 Construction Limitations Category C - Masonry components shall be constructed to conform to the limitations of this Section.

(a) Reinforcement:

All masonry shall be reinforced in accordance with Section 4A.

(b) Tie Anchorages:

In addition to the requirements of Clause 2.426.2 (b) for Category B a minimum of 135 degrees plus an extension of at least 6 tie diameters but not less than 100 mm at the tie shall be provided.

Such ties shall be located within the top 100 mm of the member and shall consist of not less than 12 mm diameter or three 10 mm ties.

(c) Reinforced Columns:

In addition to the requirements of Section 4A, sub-section 2.413 for stiffener columns or reinforced masonry columns, no longitudinal bar shall be farther than 150 mm from a laterally supported bar.

Except at corner bars, ties providing lateral support may be in the form of cross-ties engaging bars at opposite sides of the column.

The tie spacing shall be not greater than 16 bar diameters or 200 mm for the full height of masonry shear wall boundary columns and all other columns stressed by tensile or compressive axial over-turning forces due to seismic effects and for the tops and bottoms of all other columns for a distance of 1/6 of clear height but not less than 450 mm nor the maximum column dimension. The spacing for the remaining column height shall be not greater than 16 bar diameters, 48 tie diameters or the least column dimension, but not more than 450 mm.

(d) Shear Walls and Shear Wall Boundary Elements:

Shear walls and boundary elements shall conform to the requirments of sub-section 2.429.

As applicable, boundary members shall also conform to one of the following:

1. Part 2 Sections 6 or 7.

   When of reinforced concrete or structural steel.

2. Part 2 Section 4A sub-section 2.413.

   When of masonry.

(e) Joint Reinforcement (Special Masonry Type)

Longitudinal masonry joint reinforcement may be used in reinforced grouted masonry and reinforced hollow unit masonry only to fulfill minimum reinforcement rations but shall not be considered in the determination of the strength of the member.

(f) Stacked Bond Construction:

The minimum ratio of horizontal reinforcement shall be 0.0015 for structural walls of stacked bond construction.

Units shall have at least one end open and shall be fully grouted at the head joint.

Except for cases falling under Section 4C, the maximum spacing of horizontal reinforcing shall not exceed 600 mm. Where reinforced hollow unit construction forms part of the seismic resisting system, the construction shall be grouted solid and all head joints shall be made through the use of open-end units.

(g) Non-structural Components:

Non-structural walls, partitions and components shall be designed to support themselves and to resist seismic forces induced by their own weight.

Holes and openings shall be suitably stiffened and strengthened. Non-structural walls and partitions shall be anchored in accordance with the requirements of Section 4A.

2.427.3 MATERIALS LIMITATIONS - The following materials shall not be used unreinforced for any non-structural purpose apart from the exceptions proposed in Section 4C or Part 5 of this Code.

MATERIALS	STANDARD DESIGNATION
(Unreinforced) Unburned clay masonry (e.g. stabilised soil)	(Uniform Building Code USA Standard 24-15)
(Unreinforced) Structural clay Load-bearing wall Tile	ASTM C34, C212

2.427.4 The following materials shall not be used unreinforced for any structural purpose apart from the exceptions shown in Section 4C and Part 5.

MATERIALS	STANDARD DESIGNATION
Hollow load-bearing concrete masonry units or with mortar of compressive strength less than 12.6 N/mm^2	ASTM C90
Building brick and hollow brick made from clay or shale of Grade NW	ASTM C62, C216, C652

2.428 Construction and Material Limitations for Seismic Performance Category D

2.428.1 Buildings assigned to Category D shall conform to all of the requirements for Category C and to the additional requirements and limitations of this Section.

2.428.2 CONSTRUCTION LIMITATIONS CATEGORY D - Materials for mortar and grout for structural masonry shall be measured in suitable calibrated devices. Shovel measurements are not acceptable. Where considerable shrinkage is anticipated an approved admixture of a type that reduces early water loss and produces a net expansion action shall be used for grout for structural masonry. The thickness of the grout between masonry units and reinforcing shall be a minimum of 12 mm for structural masonry.

(a) Reinforced Hollow Unit Masonry:

Structural reinforced hollow unit masonry shall conform to requirements below:

1. Wythes and elements shall be at least 200 mm in nominal thickness with clear, unobstructed continuous vertical cells, without offsets, large

enough to enclose a circle of at least 90 mm in diameter and with a minimum area of 9,700 mm^2.

2. All grout shall be coarse grout. Consolidation shall be by mechanical vibration only. All grout shall be reconsolidated after excess moisture has been absorbed but before work-ability has been lost.

   NOTE: Concrete of minimum crushing strength 15 N/mm^2 is acceptable with conventional water cement ratios, (e.g. 0.50 - 0.6) and compacted manually, provided that cavities are filled at intervals not exceeding 600 mm in height.

3. Vertical reinforcement shall be securely held in position at tops, bottoms, splices, and at intervals not exceeding 112 bar diameters. Horizontal wall reinforcement shall be securely tied to the vertical reinforcement or held in place during grouting by equivalent means.

4. Minimum nominal column dimension shall be 300 mm.

(b) Stack Bond Construction:

Horizontal reinforcement shall be 0.0015 of wall cross-sectional area for non-structural and 0.0025 for structural masonry. Maximum spacing of bars shall be 600 mm and 400 mm respectively.

2.428.3 MATERIAL LIMITATIONS - The following materials shall not be used for any structural or non-structural masonry apart from those exceptions set out in Section 4C:

(i) All materials listed in this Section for Categories B AND C.
(ii) Hollow non-load-bearing concrete masonry units to ASTM C129 shall not be used.
(iii) Sand-lime building brick to ASTM C73 shall not be used for any structural masonry.

2.428.4 SPECIAL INSPECTION - Special inspection in accordance with Appendix 2 shall be provided for all structural masonry.

2.429 Shear Wall Requirements

2.429.1 Shear walls shall comply with the requirements of this Section.

2.429.2 REINFORCEMENT - The minimum ratio of reinforcement for shear walls shall be 0.0015 in each direction. The maximum spacing of reinforcement in each direction shall be the smaller of the following dimensions: one-third of the length and height of the element but not more than 800 mm. The area and spacing of reinforcement perpendicular to the shear reinforcement shall be at least equal to that of the required shear reinforcement. The portion of the reinforcement required to resist shear shall be uniformly spaced.

Exception: For shear walls constructed using running bond, the ratio of reinforcement may be decreased to 0.0007 provided that all shear is resisted by the reinforcement. The sum of the ratios of horizontal and vertical reinforcement shall not be less than 0.002.

2.429.3 BOUNDARY MEMBERS - Where cross walls or boundary members form a part of the shear wall system, the intersections shall be constructed as required for the walls themselves. Connections to concrete shall conform to Section 4A.

2.429.4 Where boundary members are of structural steel, the shear transfer between the walls and the boundary member shall be developed by fully encasing the element in grout, dowels, bolts, or shear lugs, or by similar approved methods.

2.429.5 When the structural system, as described in Part 2 Section 3 of this Code is a Building Frame System or a Dual System, (i.e. involving both frames and walls) boundary members shall be provided at each end of the wall. In such cases members shall be of the same construction as the frame columns.

2.429.6 Where the frame is a ductile moment resisting frame, those columns shall conform to the requirements for such members in Part 2 Section 3 and 6 or 7 of this Code as applicable.

2.429.7 Shear walls in buildings with a bearing wall system having design compressive stresses in excess of 0.2 f_m', as calculated, for any load combination including earthquake effects, shall have vertical boundary members along the wall edges as described in Part 2 Section 3.

2.429.8 The required vertical boundary members and such other similar vertical elements as may be required shall be designed to carry all the vertical forces resulting from the wall loads, the tributary dead and live loads, and the seismic forces prescribed in these provisions.

2.429.9 Horizontal reinforcing in the walls shall be anchored to the vertical elements. Where the boundary element is structural steel this shall be accomplished by welding or by extension, with bends if required, into grout fully surrounding the column.

2.429.10 COMPRESSIVE STRESSES - For loading combinations including in-plane seismic forces, allowable compression stresses at any point shall not exceed those allowed for axial compression. The allowable working stress values for reinforced masonry shall be the allowable working stresses given in Table 2.416.1 and applicable reductions for slenderness effects shall apply. The minimum horizontal distance between lateral supports may be considered for walls as well as the minimum vertical distance.

Exception: For pier type wall elements that do not extend from floor to floor compression stresses under combined loading at any point may be limited to those allowed for flexural compression provided that Formula in clause 2.416.4 is also satisfied.

2.429.11 HORIZONTAL COMPONENTS - When shear reinforcing is required for loads that include seismic effects and diagonal bars are not provided, reinforcement approximately perpendicular to the required shear reinforcement shall be provided equal in amount and spaced not further apart than is required for shear reinforcing. Horizontal reinforcing shall anchor into or be continous through the pier elements. Horizontal components may be separated from the shear wall system by means of true joints. The joints shall provide for building movement determined in accordance with Part 2 Section 3. The horizontal components shall be anchored to the building and designed as otherwise required by these provisions.

2.430 Strength of Members and Connections

2.430.1 The strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor Ø, and material strength of 2.5 times the allowable working stresses of Table 2.416.1 in Section 4A. The value of Ø shall be as follows:

$\emptyset = 1.0$ When considering axial or flexural compression and bearing stresses in the masonry.

$\emptyset = 0.8$ For reinforcement stresses except when considering shear.

ϕ = 0.6	When considering shear carried by shear reinforcement and blocks.
ϕ = 0.6	When considering masonry tension parallel to the bed joints i.e. horizontally in normal construction.
ϕ = 0.4	When considering shear carried by the masonry.
ϕ = Zero	When considering masonry tension perpendicular to the bed joints, i.e. normal construction.

2.431 **Structural Design Requirements**

2.432 **Design Basis**

2.432.1 The requirements for sub-sections 2.431 through 2.438 shall control the selection of the seismic analysis procedures to be used in the design of buildings and their components. The design seismic forces, and their distribution over the height of the building, shall be established in accordance with the procedures in sub-section 2.439 or an approved alternate procedure. The internal forces in the members of the building shall be determined using a linearly elastic model. Individual members shall be sized for the shears, axial forces, and moments determined in accordance with these provisions, and connections shall develop the strength of the connected members or the forces indicated above. The deformation of the building shall not exceed the prescribed limits when the building is subjected to the design seismic forces.

2.432.2 A continous load path, or paths with adequate strength and stiffness, shall be provided, which will transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to accommodate the forces developed or the movements imparted to the building by the design ground motions. In the determination of the foundation design criteria, special recognition shall be given to the dynamic nature of the forces, the expected ground motions, and design basis for strength and ductility of the structure.

2.433 **Site Effects**

2.433.1 Soil types and site coefficient, S, are given in this sub-section.

2.433.2 SOIL PROFILE TYPES - The effects of site conditions on building response shall be established based on soil profile types defined as follows:

(a) Soil Profile S_1 - is a profile with:

Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 760 metres/second or other approved means of classification, or

Still soil conditions where the soil depth is less than 60 metres and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

(b) Soil Profile Type S_2 - is a profile with:

Deep cohesionless or stiff clay conditions, including sites where the soil depth exceeds 60m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

(c) Soil Profile Type S_3 - is a profile with:

Soft-to-medium-stiff clays and sands, characterized by 9m or more of soft-to-medium-stiff clays without intervening layers of sand or other cohesionless soils.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile S_2 shall be used.

2.433.3 SITE COEFFICIENT - S is a coefficient for the effects of the site conditions on building response and is given in Table 2.438.1

2.434 Framing Systems

2.434.1 Three types of general framing systems utilising masonary walls are recognized for purposes of these provisions as shown in Table 2.438.2. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in sub-section 2.437 and in Section 4B for buildings assigned to the various seismic preformance categories.

2.434.2 CLASSIFICATION OF FRAMING SYSTEMS - Each building, or portion thereof, shall be classified as one of the three general framing system - types of Table 2.438.2 utilising masonry walls. The response modification factor, R, and the deflection amplification factor, C_d, are given in Table 2.438.2 and are used in determining the base shear and the design storey drift.

2.434.3 COMBINATIONS OF FRAMING SYSTEMS - Where combinations of framing systems are incorporated into the same building the following requirements shall be fulfilled:

(a) R Value: The value of R in the direction under consideration at any level shall not exceed the lowest value of R obtained from Table 2.438.2 for the seismic resisting system in the same direction considered above that level.

Exception: This requirement need not apply to supported systems with a weight equal to or less than 10 percent of the weight of the building.

(b) Detailing Requirements:

For components common to systems having different R values, the detailing requirements which are required by the higher R value shall be used.

2.434.4 FRAMING SYSTEMS FOR SEISMIC PERFORMANCE CATEGORY A AND B - Any type of building framing system permitted in these provisions may be used for buildings assigned to Categories A and B.

2.434.5 FRAMING SYSTEMS FOR SEISMIC PERFORMANCE CATEGORY C - Buildings assigned to Category C shall conform to the framing system requirements for Category B and to the additional requirements and limitations of this Section.

(a) Seismic Resisting Systems for Performance Category C: Seismic resisting systems in building over 49m in height shall be one of the following :

1. Moment resisting system with Special Moment Frames.

2. A Dual System

3. A system with structural steel or cast-in-place concrete braced frames or shear walls so arranged that braced frames or walls in any plane resist no more than 33 percent of the seismic design force including torsional effects; this system is limited to buildings not over 73m in height.

(b) Interaction Effects:

Moment resisting space frames which are enclosed by, or adjoined by, more rigid elements not considered to be part of the seismic resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic-force-resisting capability of the space frame. The design shall consider and provide for the effect of these rigid elements on the structural system at building deformations corresponding to the design storey drift $\triangle$ as determined by sub-section 2.444.

(c) Deformation Compatibility:

All structural elements not considered in the design to be part of the seismic resisting system shall be investigated and shown to be adequate for the vertical load-carrying capacity and the induced moments resulting from the design storey drift $\triangle$ as determined in accordance with sub-section 2.444.

(d) Special Moment Frames:

A Special Moment Frame which is used, but not required by these provisions, may be discontinued and supported by a more rigid system with a lower R value subject to the requirements in sub-section 2.437.

A Special Moment Frame which is required by these provisions shall be continued down to the foundation.

2.434.6 FRAMING SYSTEMS FOR SEISMIC PERFORMANCE CATEGORY D - The framing systems of buildings assigned to Category D shall conform to the requirements and limitations of this Section.

The height limitations of clause 2.434.5 shall be reduced from 49m to 30m and for braced frame or shear wall systems the maximum height shall be reduced from 73m to 49m.

2.435 Building Configuration

2.435.1 For purposes of seismic design, buildings shall be classified as regular or irregular as specified in this Section. Both plan and vertical configurations of a building shall be considered when determining whether a building is to be classified as regular or irregular.

2.435.2 Buildings which have an approximately symmetrical geometric configuration and which have the building mass and seismic resisting system nearly coincident shall be classified as regular.

2.435.3 PLAN CONFIGURATION - For purposes of determining diaphragm component forces and distribution of seismic forces to vertical components of the seismic resisting system, a building shall be classified as irregular when any of the following occurs:

(a) The building does not have an approximately symmetrical geometric configuration or has re-entrant corners with significant dimensions.

(b) There is the potential for large torsional moments because there is significant eccentricity between the seismic resisting system and the mass tributary to any level.

(c) The diaphragm at any single level has significant changes in strength or stiffness.

2.435.4 VERTICAL CONFIGURATION -For purposes of selecting an analysis procedure for determining seismic forces and the distribution of these forces, a building shall be classified as irregular when any of the following occurs:

(a) The building does not have an approximately symmetrical geometric configuration about the vertical axes or has horizontal offsets with significant dimensions.

(b) The mass-stiffness ratios between adjacent stories varies significantly.

2.436 Analysis Procedures

2.436.1 This sub-section prescribes the minimum analysis procedure to be followed. A more rigorous generally accepted procedure may be used in lieu of the minimum applicable procedure. In no case shall the alternate procedure use fundamental building periods greater than permitted in sub-sections 2.439 through 2.440.

2.436.2 ANALYSIS PROCEDURES FOR SEISMIC PERFORMANCE CATEGORY - Regular or irregular buildings assigned to Category A need not be analyzed for seismic forces for the building as a whole. The provisions of sub-section 2.437 shall apply to the components indicated therein.

2.436.3 ANALYSIS PROCEDURES FOR SEISMIC PERFORMANCE CATEGORY B - Regular or irregular buildings assigned to Category B shall be as a minimum analyzed in sub-sections 2.439 through 2.444.

Exception: In buildings assigned to Category B the design seismic forces may be applied separately in each of the two orthogonal directions.

2.436.4 ANALYSIS PROCEDURES FOR SEISMIC PERFORMANCE CATEGORY C AND D - Buildings classified as regular and assigned to

Category C or D shall, as a minimum, be analyzed in accordance with the procedures given in sub-sections 2.439 through 2.444. Buildings classified as irregular, should be avoided, if at all possible. Where unavoidable they shall be analyzed with special consideration for the dynamic characteristics of the building.

2.437 **Design and Detailing Requirements**

2.437.1 The design and detailing of comoponents of the seismic resisting system and of other structural and nonstructural components shall be as specified in this sub-section.

2.437.2 DESIGN AND DETAILING REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY A - Buildings assigned to Category A may be constructed using any material permitted in Section 4B. These buildings need only comply with the minimum seismic force requirements of clauses 2.438.6 through 2.438.9 requiring adequate ties, continuity and anchorage for all portions of the building. They shall also comply with Part 2 Section 5 - Foundations.

2.437.3 DESIGN AND DETAILING REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY B - Buildings assigned to Category B shall conform to the requirements and limitations.

(a) Components: Components of the seismic and other structural components shall conform to the requirements of sub-section 2.438 (except clause 2.438.13) concerning ties, continuity and anchorage and to Part 2 Section 5 Concerning Foundation Requirements.

(b) Materials: The materials, and the systems composed of those materials, shall conform to the requirements and limitations in sub-section 2.426 for Category B.

(c) Openings: Where openings occur in shear walls or diaphragms or other plate-like elements, chords shall be provided at the edges of the openings to resist the local stress created by the opening. These chords shall extend into the body of the wall or diaphragm a distance sufficient to develop and distribute the stress of the chord member.

2.437.4 DESIGN AND DETAILING REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY C - Buildings assigned to Category C shall conform to the requirements for Category B and to the following requirements and limitations.

(a) Components: Components of the seismic resisting system and other structural components shall also conform to the requirements of Clause 2.438.15 Part 2 Section 5 regarding Foundations.

(b) Materials: The materials and systems composed of these materials, shall conform to the requirements and limitations in sub-sections 2.421 through 2.430 for Category C.

2.437.5 DESIGN AND DETAILING REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY D - Buildings assigned to Category D shall conform to the following requirements and limitations.

2.437.6 The materials, and the systems composed of those materials, shall conform to the requirements and limitations of Part 2 Section 5 concerning Foundations and sub-sections 2.421 through 2.430 concerning construction and materials for Category D.

2.438 Structural Component Load Effects

2.438.1 All building components shall be provided with strengths sufficient to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead and live loads. The direction of application of seismic forces used in design shall be that which will produce the most critical load effect in each component. The second order effects shall be included where applicable.

2.438.2 COMBINATION OF LOAD EFFECTS - The effects on the building and its components due to gravity loads and seismic forces shall be combined in accordance with Formula 2.438.2.1 or, as applicable, 2.438.2.2 or 2.438.2.3.
Combination of load effects =

$$1.2\ Q_D + 1.0\ Q_L \pm 1.0\ Q_E \qquad 2.438.2.1$$

Combination of load effects =

$$0.8Q_L \pm 1.0\ Q_E \qquad 2.438.2.2$$

For partial penetration welded steel column splices or for unreinforced masonry and other brittle materials, systems, and connections:

Combination of load effects =

$$0.5\ Q_D \pm 1.0\ Q_E \qquad 2.438.2.3$$

2.438.3 ORTHOGONAL EFFECTS - The critical load effect due to direction of application of seismic forces on the building may be assumed to be satisfied if components and their foundations are desgined for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used.

2.438.4 DISCONTINUITIES IN STRENGTH OF VERTICAL RESISTING SYSTEM - The design of a building shall consider the potentially adverse effect when the ratio of the strength provided in any storey to the strength required is significantly less than that ratio for the storey immediately above and the strengths shall be adjusted to compensate for this effect.

2.438.5 NON-REDUNDANT SYSTEMS - The design of a building shall consider the potentially adverse effect that the failure of a single member, connection, or component would have on the stability of the building and appropriate design modifications shall be made to mitigate this effect.

2.438.6 TIES AND CONTINUITY - All parts of the building shall be interconnected and the connections shall be capable of transmitting the seismic force F_p induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist $A_v/3$* times the weigth of the smaller portion but not less than 5 percent of the portion's weight.

2.438.7 As a minimum a positive connection for resisting a horizontal force shall be provided for each beam, girder, or truss to its support which shall have a minumum strength acting along the span of the member equal to 5 percent of the dead and live load reaction.

2.438.8 CONCRETE OR MASONRY WALL ANCHORAGE - Concrete and masonry walls shall be anchored to the roof and all floors which provide lateral support for the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting a seismic lateral force, F_p, induced by the wall but not less than a force of 1,460 N/m of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 m.

2.438.9 ANCHORAGE OF NON-STRUCTURAL SYSTEMS - When required by Part 2 Section 3 all portions or components of the building shall be anchored, for the seismic force, prescribed therein.

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* For the purpose of this Section, A_v can be taken equal to A_a from Table 2.424.1.

2.438.10 COLLECTOR ELEMENTS - Collector elements shall be provided which are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces.

2.438.11 DIAPHRAGMS - The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the building.

2.438.12 Floor and roof diaphragms shall be designed to resist the seismic forces determined as follows:

A minimum force equal to 0.5 A_v * times the weight of the diaphragm and other elements of the building attached thereto plus the portion of the V_x required to be transferred to the components of the vertical seismic resisting system because of offsets or changes in stiffness of the vertical components above and below the diaphragm.

2.438.13 Bearing Walls - Exterior and interior walls and their anchorage shall be designed for a force * A_v W_c normal to the flat surface with a minimum of 0.1 W_c.

Interconnection of dependent wall elements and connections to supporting framing systems shall have sufficient ductility or rotational capacity, or have sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

2.438.14 INVERTED PENDULUM-TYPE STRUCTURES - Inverted pendulum-type structures where the seismic resisting system acts essentially as an isolated cantilever(s). Supporting columns or piers of inverted pendulum-type structures shall be designed for the bending moment calculated at the base determined using the procedures given in Sub-section 2.440 and varying uniformly to a moment at the top equal to one half the calculated bending moment at the base.

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* For the purpose of this Section A_v can be taken equal to A_a from Table 2.424.1.

2.438.15 VERTICAL SEISMIC MOTIONS FOR BUILDINGS ASSIGNED TO CATEGORIES C AND D - The vertical component of earthquake motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. For horizontal cantilever components, these effects may be satisfied by designing for a net upward force of $0.2Q_D$. For other horizontal components employing prestressing these effects may be satisfied by Formula 2.438.2.3

2.438.16 DEFLECTION AND DRIFT LIMITS - All portions of the building shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection, Q_x (as determined in sub-section 2.444), corresponding to the seismic design forces.

2.438.17 The design storey drift, Δ, as determined in sub-section 2.444 shall not exceed the allowable storey drift, Δ_a, obtained from Table 2.438.3 for any storey.

TABLE 2.438.1
SOIL PROFILE COEFFICIENT

Soil Profile Type:	S_1	S_2	S_3
Soil Profile coefficient (S):	1.0	1.2	1.5

TABLE 2.438.2
RESPONSE MODIFICATION[1]
COEFFICIENTS

Type of Structural System	Vertical Seismic Resisting System	Coefficients $R^{(3)}$	Coefficients $C_d^{(4)}$
Bearing Wall System: A structural system with bearing walls providing support for all, or major portions of, the vertical loads.	Light framed walls with shear panels	6 1/2	4
Seismic force resistance is provided by shear walls or braced frames.	Shear Walls:		
	Reinforced concrete	4 1/2	4
	Reinforced masonry	3	3
	Braced frames	4	3 1/2
	Unreinforced and partially reinforced masonry shear walls[2]	1 1/4	1 1/4

Dual System: A structural system with an essentially complete space frame providing support for vertical loads	Shear Walls: Reinforced concrete	8	6 1/2
	Reinforced masonry	6 1/2	5 1/2
A special moment frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces	Wood sheathed shear panels	8	5
The total seismic force resistance is proveded by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities	Braced	6	5

1. These values are based on the Draft ATC 3. They are the best data available at the time of writing but will need to be reviewed periodically.

2. Unreinforced masonry is not permitted for portions of buildings assigned to Category B. Unreinforced or partially reinforced masonry is not permitted for buildings assigned to Categories C and D; see Section 4B, sub-sections 2.425 through 2.430. Exceptions to these rules are as set out in Section 4C for single storey buildings.

3. R is coefficient for use in Formula 2.440.2 and 2.440.3 sub-section 2.440.

4. C_d is coefficient for use in Formula 2.444.1.

TABLE 2.438.3
ALLOWABLE STOREY DRIFT Δ_a

IMPORTANCE FACTORS

Importance Factors:	1.5	1.2	1.0
Δ_a:	0.010 h_{sx}	0.015 h_{sx}	0.015 h_{sx}

Where there are no brittle-type finishes in buildings three storeys or less in height, these limits may be increased one-third.

2.439 **Equivalent Lateral Force Procedure**

2.439.1 GENERAL - The requirements of sub-sections 2.439 through 2.444 shall control the seismic analysis of buildings prescribed in sub-sections 2.431 through 2.438.

2.440 **Seismic Base Shear**

2.440.1 The building, considered to be fixed at the base, shall be designed to resist the lateral seismic base shear, V, in the direction being analysed as determined in accordance with the following formula:

$$V = C_s W \qquad 2.440.1$$

where: C_s = the seismic design coefficient.

W = the total gravity load of the building. W shall be taken equal to the total weight of the structure and applicable portions of other components including, but not limited to, the following:

(a) Partitions and permanent equipment including operating contents.

(b) For storage and warehouse structures, a minimum of 25 percent of the floor live load.

2.440.2 The value of C_s may be determined in accordance with Formula 2.440.2 or 2.440.3. Formula 2.440.2 requires calculation of the fundamental period of the building as specified in Cl. 2.440.6. If the period of the building is not calculated, Cs shall be determined using Formula 2.440.3 or 2.440.4 as appropriate.

2.440.3 CALCULATION OF SEISMIC COEFFICIENT - When the period of the building is computed, the seismic coefficient C_s shall be determined with the following formula:

$$C_s = \frac{1.2A_vS}{RT^{2/3}} \qquad 2.440.2$$

where:

A_v = the coefficient representing effective peak velocity related acceleration. For the purposes of this Section, A_v can be taken equal to the values for A_a in Table 2.424.2.

S = the coefficient for the soil profile characteristics of the site as given in Table 2.438.1.

R = the response modification factor as given in Table 2.438.2

T = the fundamental period of the buildign as determined in Clause 2.440.6.

2.440.4 C_s need not be taken as greater than the value given by Formula 2.440.3 and 2.440.4.

2.440.5 For the design of a building where the period is not calculated the value of C_s shall be determined in accordance with the following formula:

$$C_s = 2.5\ A_a/R \qquad 2.440.3$$

where:

A_a = the seismic coefficient representing the effecitive peak acceleration as indicated in Table 2.424.2.

Exception: For soil profile type S_3 in areas where A_a = 0.30, C_s shall be determined in accordance with the following formula:

$$C_s = 2\ A_a/R \qquad 2.440.4$$

2.440.6 PERIOD DETERMINATION - The fundamental period of the building, T, in Formula 2.440.2 may be determined based on the properties of the seismic resisting system in the direction being analysed and the use of established methods of mechanics assuming the base of the building to be fixed but shall not exceed 1.2 T_a. Alternatively the value of T may be taken equal to the approximate fundamental period of the building, T_a, used to establish a minimum seismic base shear for the building and determined in accordance with one of the following formulas.

2.440.7 For moment-resisting structures where the frames are not enclosed or adjoined by more rigid components tending to prevent the frames from deflecting when subjected to seismic forces:

$$T_a = C_T h_n^{3/4} \qquad 2.440.5$$

where

C_T = 0.035 for steel frames

C_T = 0.025 for concrete frames

h_n = the height in feet above the base to the highest level of the building.

2.440.8 For all other buildings: e.g. masonry buildings as covered by this Section:

$$T_a = \frac{0.05h_n}{\sqrt{L}} \qquad 2.440.6$$

where L = the overall length of the building at the base in the direction under consideration.

2.441 **Vertical Distribution of Seismic Forces**

2.441.1 The lateral seismic shear force, F_x, induced at any level, shall be determined in accordance with the following formula:

$$F_x = C_{vx}V \qquad 2.441.1$$

where

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad 2.441.2$$

k is a coefficient related to the building period as follows:

For buildings having a period of 0.5 seconds or less, k = 1.

For buildings having a period of 2.5 seconds or more, k = 2.

For buildings having a period between 0.5 and 2.5 seconds, k may be taken as 2 or may be determined by linear interpolation between 1 and 2.

w_i, w_x = the portion of w located at or assigned to level i or x.

h_i, h_x = the height above the base level i or x.

2.442 **Horizontal Shear Distribution and Torsion**

2.442.1 The seismic shear force at any level, V_x, shall be determined in accordance with the following formula:

$$V_x = \sum_{i=x}^{n} F_i \qquad 2.442.1$$

2.442.2 The force, V_x, and the associated torsional forces shall be distributed to the various vertical components of the seismic resisting system in the storey below level x with due consideration given to the relative stiffnesses of the vertical components and the diaphragm.

2.442.3 The design shall provide for the torsional moment M_t resulting from the location of the building masses plus the torsional moments M_{ta} caused by assumed displacement of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the building perpendicular to the direction of the applied forces.

2.443 **Overturning**

2.443.1 Every building shall be designed to resist overturning effects caused by the seismic forces. At any level, the increment of overturning moment in the storey under consideration shall be distributed to the various walls or frames in the same proportion as the distribution of the horizontal shears to those walls or frames.

2.443.2 The overturning moments shall be determined by the application of the prescribed forces as follows:

$$M_x = K\sum_{i=x}^{n} F_i \, (h_i - h_x) \qquad 2.443.1$$

where:

K = 1.0 for the top 10 stories

K = 0.8 for the 20th storey from the top and below.

K = a value between 1.0 and 0.8 determined by a straight line interpolation for stories between the 20th and 10th stories below the top.

2.443.3 The foundations of buildings, except inverted pendulum structures, may be designed for the foundation overturning design moments M_f, at the foundation-soil interface determined using Formula 2.443.6 with K = 0.75 for all building heights. The resultant seismic forces and vertical loads at the foundation-soil interface shall not fall outside the middle one-half of the base of the component(s) resisting the overturning.

2.444 **Drift Determination And P-Delta Effects**

2.444.1 Storey drifts and, where required, member forces and moments due to P-delta effects shall be determined in accordance with this sub-section.

2.444.2 STOREY DRIFT DETERMINATION - The design storey drift, $\triangle$, shall be computed as the difference of the deflections, δ_x, at the top and bottom of the storey under consideration. The deflections, δ_x shall be evaluated in accordance with the following formula:

$$\delta_x = C_d \delta_{xe} \qquad 2.444.1$$

where

C_d = the deflection amplification factor as given in Table 2.438.2

δ_{xe} = the deflections determined by an elastic analysis. The elastic analysis of the seismic resisting system shall be made using the prescribed seismic design forces (See sub-section 2.441) and considering the building fixed at the base.

2.444.3 For determining compliance with the storey drift limitation of clause 2.438.17, the deflections may be calculated as above, but the seismic resisting system and the design forces corresponding to the fundamental period of the building, T, calculated without the limit specified in clauses 2.440.6 to 2.440.8.

2.444.4 Where applicable Δ shall be increased by the incremental factor relating to the P-delta effects as determined in clauses 2.444.5 and 2.444.6 below.

2.444.5 P - DELTA EFFECTS - P-Delta effects on storey shears and moments, the resulting member forces and moments, and the storey drifts induced by these effects need not be considered when the stability coefficient, θ, as determined in accordance with Formula 2.444.2, is equal to or less than 0.10.

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad 2.444.2$$

where

Δ = the design storey drift.

V_x = the seismic shear force acting between level x and x - 1.

h_{sx} = the storey height below level x.

$P = \sum_{i=x}^{n} w_1$ ' total gravity load at and above level x

2.444.6 When θ is greater than 0.10, the incremental factor related to P-delta effects, a_d, shall be determined by rational analysis. The design storey drift determined in clause 2.444.2 shall be multiplied by the factor $(1+a_d)$ to obtain the storey drift including P-delta effects. The increase in storey shears and moments resulting from the increase in storey drift shall be added to the corresponding quantities determined without consideration of the P-delta effect.

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 4C
MINIMUM PROVISIONS FOR SINGLE STOREY HOLLOW UNIT MASONRY BUILDINGS WITH LIGHT ROOFS (AND FOR OTHER BUILDINGS OF SEISMIC PERFORMANCE CATEGORY A)

2.445 Background To Section 4C

2.445.1 Economic pressures in the Commonwealth Caribbean have caused frequent questioning of minimum reinforcement provisions, i.e. reinforcement provided where calculated stresses indicate that no steel is necessary.

2.445.2 Previous Caribbean Code efforts have followed SEAOC methods which require substantial minimum reinforcing.

2.445.3 However, these requirements were challenged for the following reasons:-

(a) SEAOC appears not to contemplate very lightly loaded walls.

(b) A few areas in the Caribbean were felt to have very low seismic Categories as defined in Section 4A.

(c) Requirements in various international codes vary considerably but research work in the United States of America suggests that minimum reinforcement requirements can be reduced.

(d) The great need for low cost housing in the Caribbean has elicited requests from policy makers that priority be placed on safety against collapse, in preference to the prevention or reduction of cracking. Section 4C is a response to the above.

2.446 Basis

2.446.1 Recommendations in this Section for Seismic Performance Categories A and B are largely based on Guidelines for One-Storey Masonry Houses in Seismic Zone 2 by Applied Technology Council-3 and US Department Housing and Urban Development.

2.446.2 Recommendations for Seismic Category C are derived from Draft Report of the Caribbean Seismic Code Committee 197 by "Caribbean Council of Engineering Organizations", and "Recommendations for Minimum Reinforcement for Single Storey Reinforced Concrete Block Walls Supporting Light Roofs": Jamaica Institution of Engineers, 197.

2.447 Scope And Applicability Of Section 4C

2.447.1 This section sets out recommendations for minimum reinforcement provisions for:

(a) Single storey single-family, residential, hollow concrete block or clay masonry buildings with light roofs.

(b) Single and two storey, single family residential buildings in hollow concrete block or clay masonry construction for Buildings in Seismic Category Type A (with respect to wall construction only).

(c) Buildings other than residential to a maximum height of 10.64 m in hollow concrete block or clay masonry construction for Buildings in Seismic Performance Category A (with respect to wall construction).

2.447.2 Reduced requirements for belt beams and stiffener columns are also described.

2.447.3 In all other respects not specifically mentioned, requirements shall be set out in Section 4A.

2.447.4 These minimum provisions will apply only where calculated reinforcement from a structural design in accordance with Subsections 2.431 through 2.438 produces even less steel.

2.447.5 It is expected that for walls fitting the description above such reinforcement can be provided without the need for structural calculations and Sections 4A and 4B can be by-passed.

2.447.6 It can be expected that these provisions will provide protection from structural failure but not from cracks and other damage and it should be noted that the walls will be more susceptible to cracks from shrinkage and minor foundation movement, than would be the case with walls designed in accordance with Section 4A and 4B.

2.448 Other Requirements For Applicability

2.448.1 The Section is applicable only to nominal dimension 150 mm wide, masonry units and to houses that are "regular" in plan and meet the following requirements.

2.448.2 Other limitations of this section are:

(a) The area under the roof must not be greater than 230 m^2.

(b) The total weight of the roofing, ceiling, insulation, and other materials supported directly by the trusses or rafters must not exceed 1.0 kN/m^2.

(c) All masonry walls must be constructed with a concrete or masonry stem wall on a concrete foundation.

(d) The unsupported height of any wall above the floor must be not greater than 2.5 m and the height of any parapet above the roof must be no greater than 1 m.

(e) The roof must be adequately braced or sarked to resist lateral forces without undue distortion and must be nailed or otherwise firmly fixed to the wood framing members (trusses or rafters).

(f) Roof trusses bearing on top of the masonry walls must not have a clear span greater 9.7 m.

(g) Roof rafters or trusses supported by a ledger bolted to the face of the masonry wall must not have a clear span greater than 5 m.

(h) Where sloping rafters are not supported by internal walls, horizontal thrust at eaves shall be resisted by ceiling ties or by other means such as belt-beams restrained by abutting interior walls.

2.449 Seismic Performance Categories

2.449.1 Seismic Performance Categories are defined in Section 4B sub-section 2.421.

2.450 Shear Panels

2.450.1 A Shear Panel is a portion or section of an exterior wall that performs the function of resisting lateral earthquakes or wind forces. At least one shear panel, a minimum of 1.8 m long and with no openings or penetrations, must be provided on each exterior wall of every house. All projecting garages or projecting wings must also have at least one shear panel on each exterior wall.

2.450.2 An unreinforced shear panel has no vertical reinforcing requirements.

2.450.3 A reinforced shear panel must have a 12 mm vertical bar in a solid grouted cell at each end of the panel. A 12mm straight dowel must be provided from the grouted cell into the stem wall at each end, and 12 mm hooked dowel must be provided from the stem wall into the foundation. Fig. 2.449.1 illustrates shear panels in a typical building.

2.451 Minimum Requirements - Single Storey Buildings For Seismic Performance Category A

2.451.1 Houses with total roof weights (including roofing, ceiling, insulaton and other materials supported by the trusses) less than or equal to 0.6 kN/m^2 may use reinforced shear panels on each elevation.

2.451.2 Houses with total roof weights greater than 0.6 kN/m^2 but less than or equal to 1.0 kN/m^2 must use reinforced shear panels on each elevation.

2.451.3 Except for verticals and dowels at jambs of large openings and reinforces shear panels, no vertical wall reinforcement is required. In lieu of reinforced belt beams, continuous horizontal joint reinforcement, No. 9 gauge wire, truss or ladder type, may be provided at the top two courses of all walls.

2.451.4 Figures 2.451.1 and 2.451.2 illustrate typical example plans and elevations and wall details in SPC 'A'.

2.451.5 These plans are illustrative only. They are intended to display the concept of shear panel size and layout.

2.452 Minimum Requirements - Single Storey Buildings in Seismic Performance Category B

2.452.1 Reinforced shear panels must be used on each elevation of every house.

2.452.2 Except for verticals and dowels at jambs of large openings and reinforced shear panels, no vertical wall reinforcement is required. In lieu of reinforced belt beams continuous horizontal joint reinformceement, No. 9 gauge wire truss or ladder type, may be provided at the top two courses of all walls.

2.452.3 Figures 2.452.1 and 2.452.2 illustrate typical example plans and elevations and wall details in SPC 'B'.

2.452.4 These plans are illustrative only. They are intended to display the concept of shear panel size and layout.

2.453 Minimum Requirements - Single Storey Buildings - Seismic Performance Category C

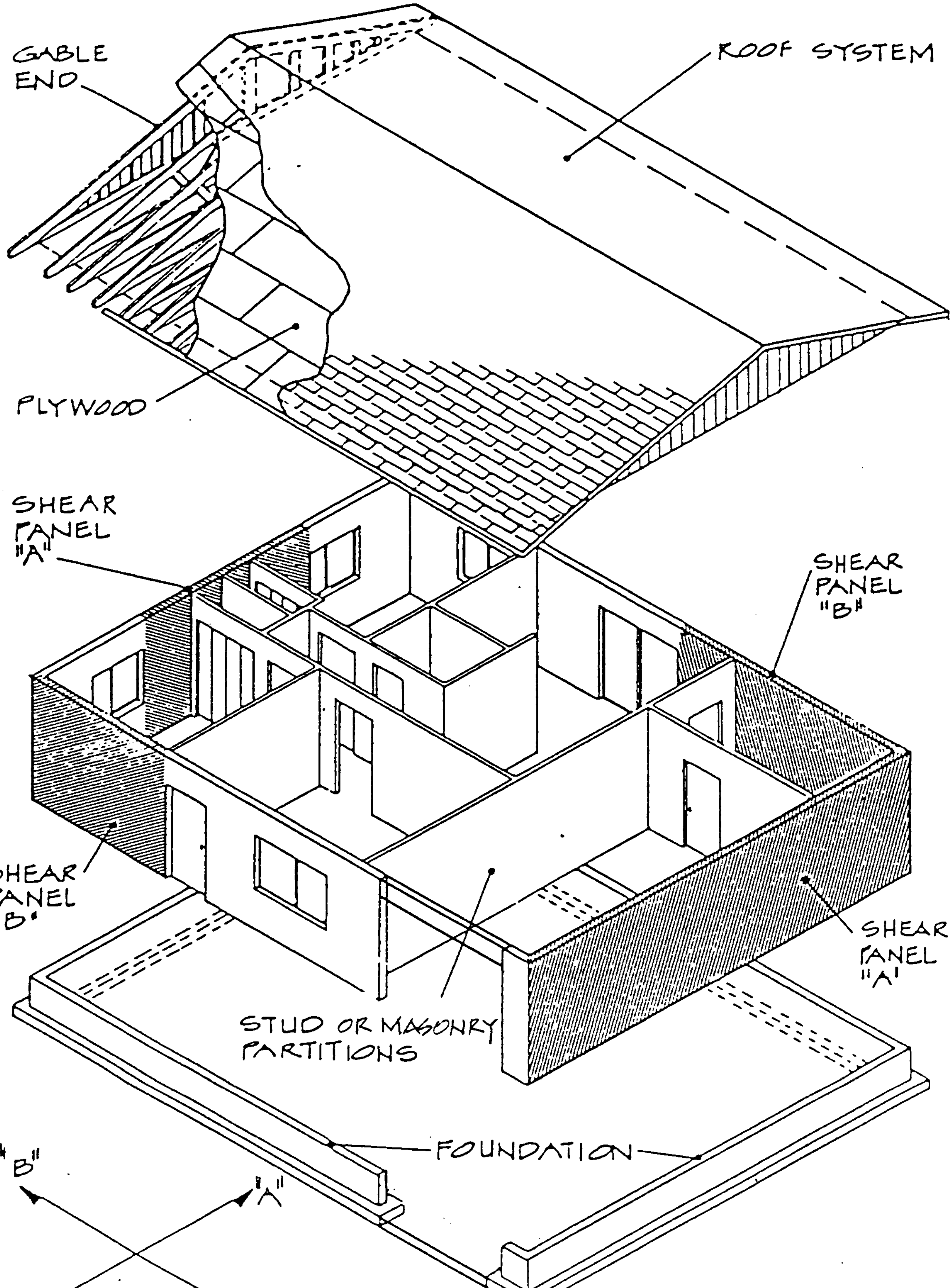
2.453.1 For Single storey hollow concrete or clay block masonry walls with light roofs of dead loading less than 1.0KN/m (e.g. timber framed roofs with clear roof spans not exceeding say 5 m or with trussed roofs not exceeding 10 m and with no horizontal thrust exerted from the roof to the walls) the following minimum provisions shall be met:

Shear Panels	- 3.0 m or 2 No 2.0m long to each exterior wall.
Vertical Steel	- 12 mm at 800 mm crs minimum.
Horizontal Steel	- Omitted if wall does not exceed 2.7 m height between foundations and eaves, otherwise, 10 mm at 800 mm.
Openings and ends	- 12 mm bar in end cavities to jambs and sills.

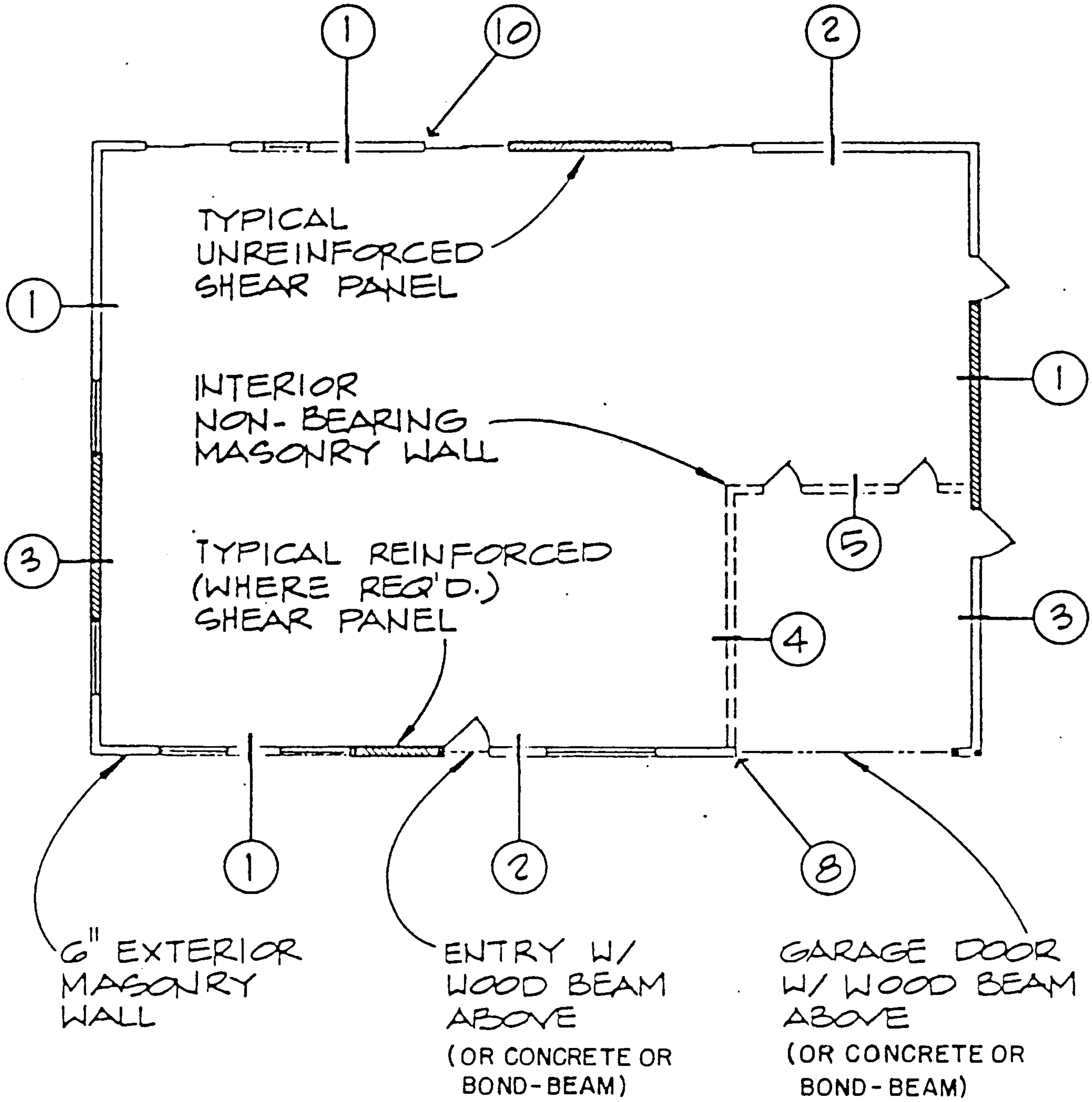
Foundations	- Top and bottom longitudinal reinforcement minimum 2-12 mm diameter pfm 2-10 mm dia top.
Corners	- If properly bonded, 3-12 mm diameter bars in corner cavities linked with 6 mm diameter bars at 400 mm crs. Otherwise provide stiffener - column in accordance with Section 4A.
Belt Beams at Top of Wall	- Minimum 200 mm deep poured concrete or bond-beam 4 No 12 mm dia bars - 10 mm links at 200 mm crs.

2.454 Hurricane Resistance

2.454.1 Where high winds are a consideration, separate attention must be given to lateral wind pressure and shear on walls and uplift on roofs for buildings in hurricane prone areas. Vertical dowel reinforcement from roof to foundation shall be not less than one 12 mm diameter bar fully grouted at 1.2 m centres, not withstanding any minimum provisions stated in the foregoing. Roofs shall be tied to the top of walls by a minimum 12 mm diameter mild steel bolts at no greater than 1.2 m centres.



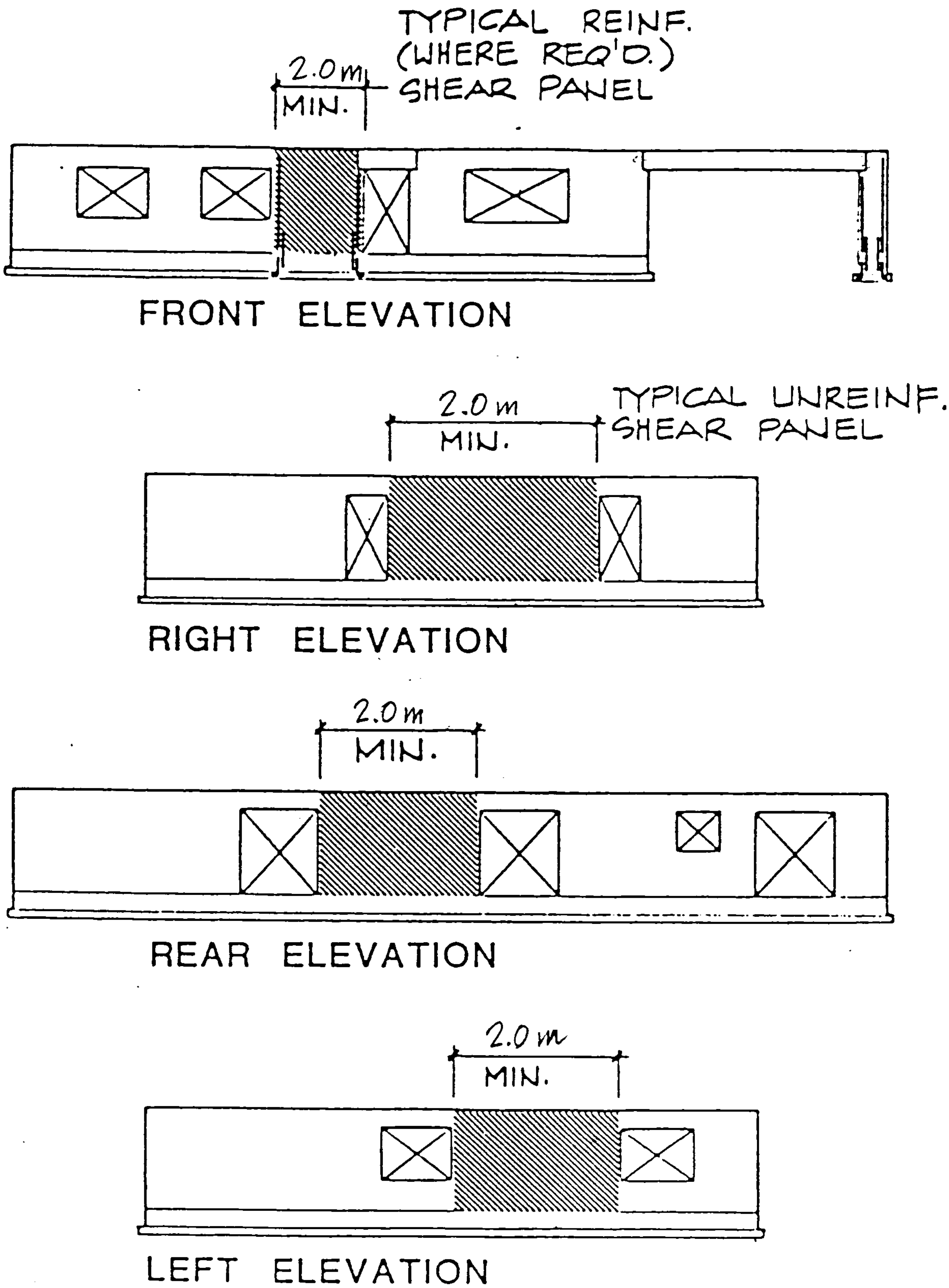
Earthquake Resistance System
(TYPICAL EXAMPLE) FIG. 2.449.1



S.P.C. - 'A'

EXAMPLE PLAN - SINGLE STOREY BUILDING IN SEISMIC PERFORMANCE CATEGORY 'A'

FIG. 2.451.1

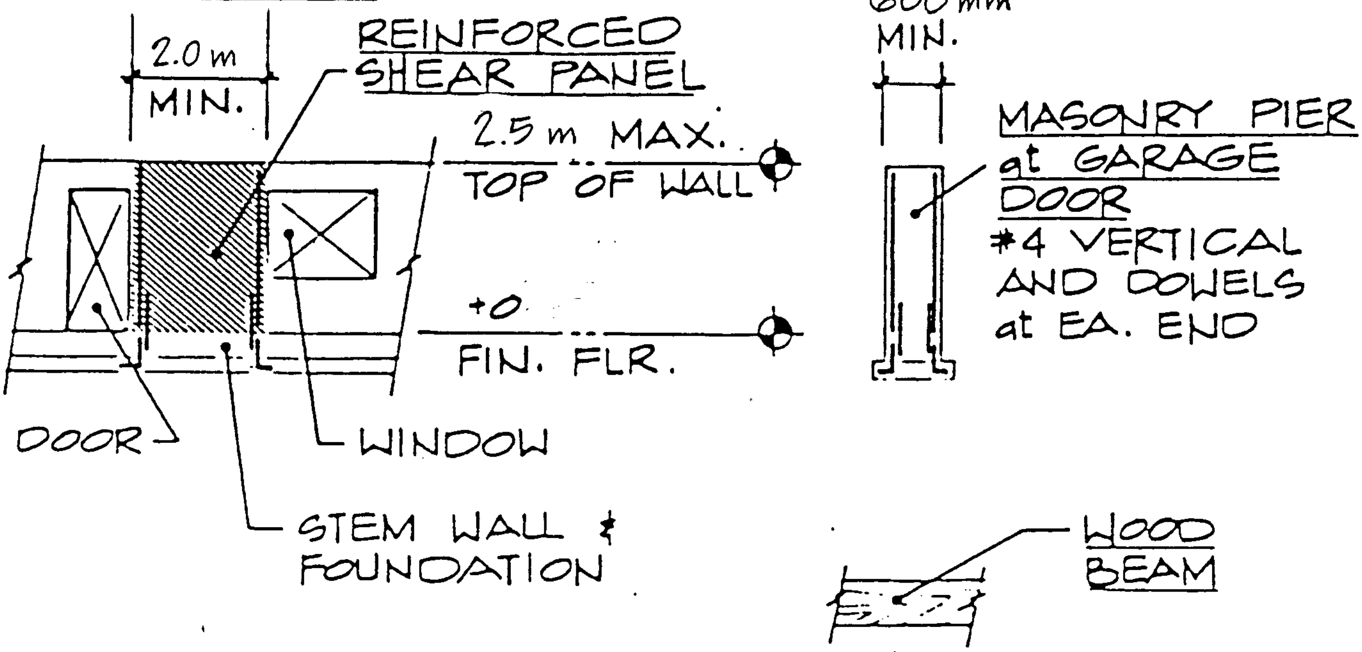


S.P.C. - 'A'

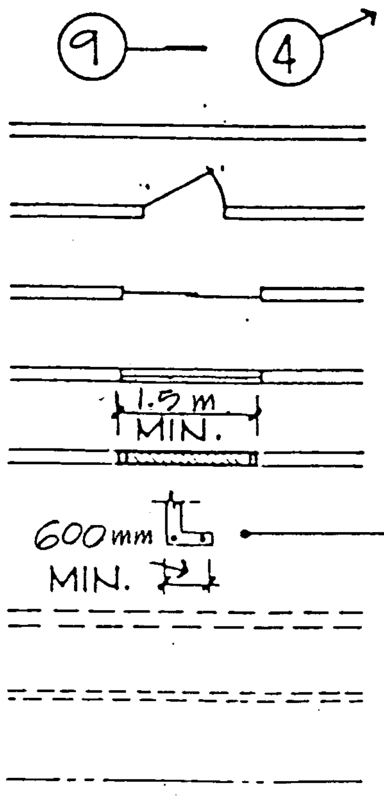
EXTERIOR ELEVATION
SINGLE STOREY

FIG. 2.451.2

Elevations



Plan

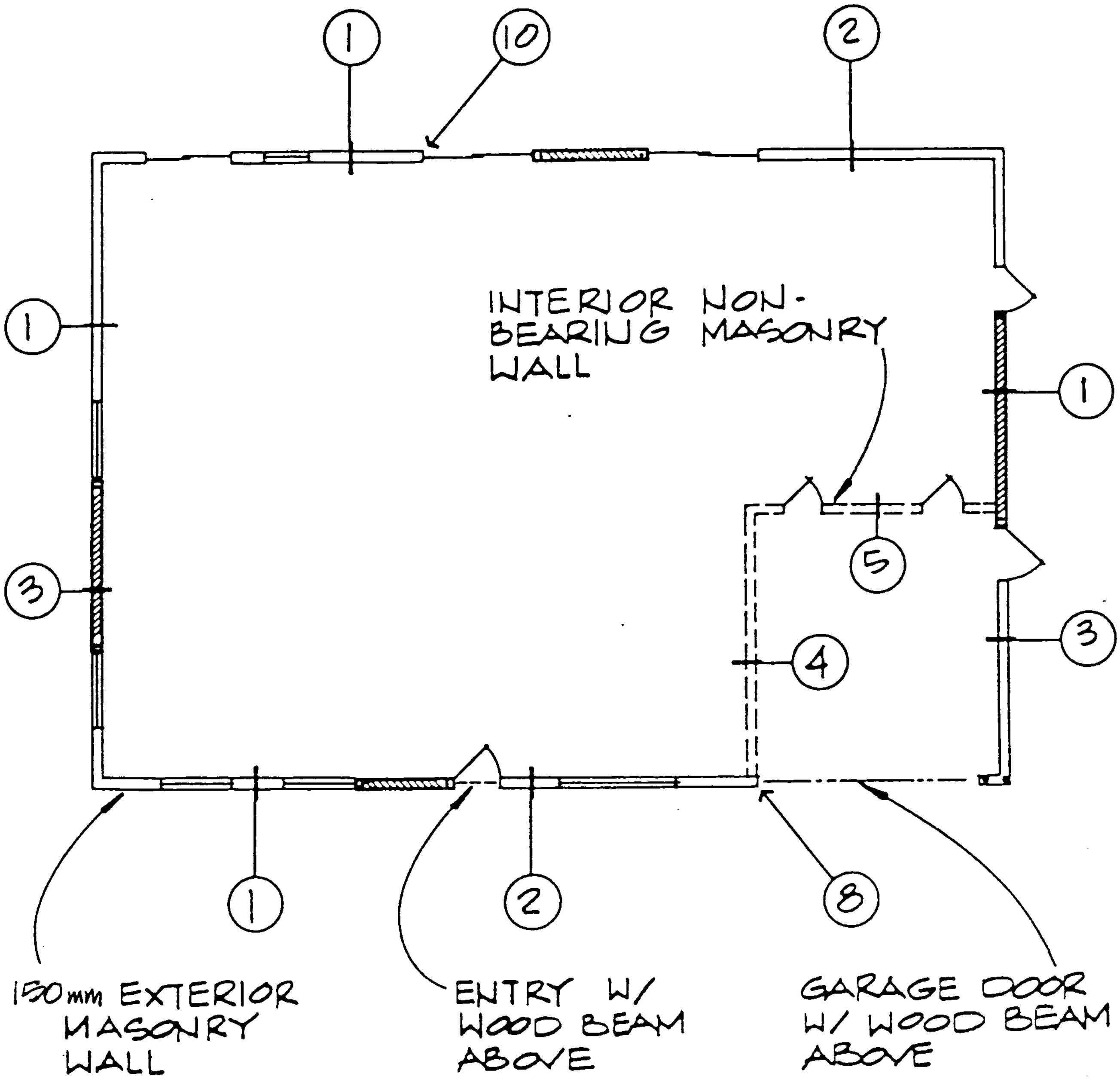


- DETAIL REFERENCE
- 150 mm EXTERIOR MASONRY WALLS
- HINGED DOOR
- SLIDING GLASS DOOR
- WINDOW
- REINFORCED SHEAR PANEL
- MASONRY PIER at GARAGE DOOR
- INTERIOR NON-BEARING MASONRY WALL
- WOOD STUD PARTITION
- WOOD BEAM

S.P.C. - 'B'

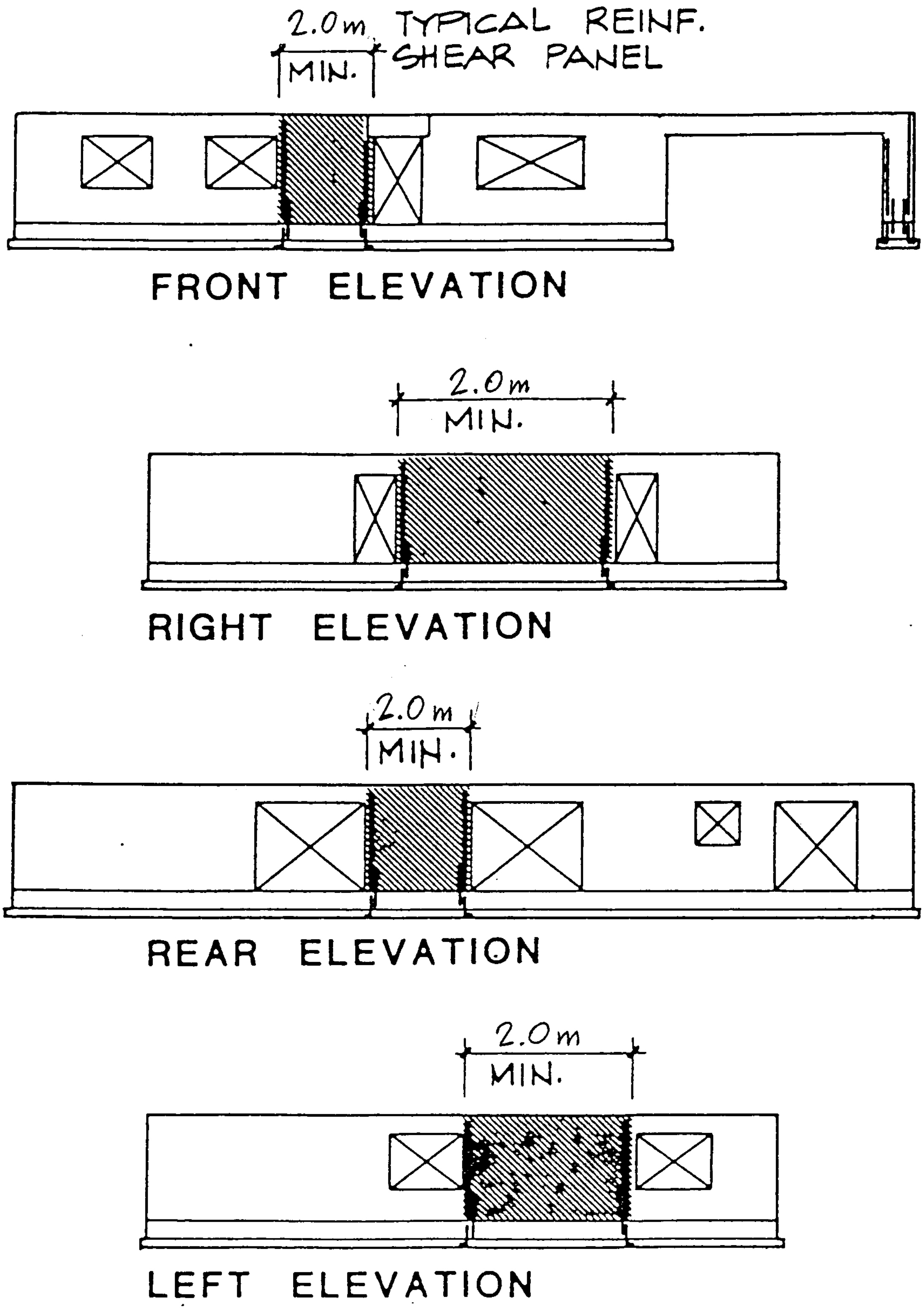
LEGEND FOR PLAN AND EXTERIOR ELEVATION - SINGLE STOREY

FIG. 2.452.1



S.P.C. - 'B' - EXAMPLE PLAN - SINGLE STOREY BUILDING IN SEISMIC PERFORMANCE CATEGORY 'B'

FIG. 2.452.2



SPC. -'B'

EXTERIOR ELEVATION - SINGLE STOREY

FIG. 2.452.3

RECOMMENDATIONS FOR SINGLE STOREY WALLS WITH LIGHT ROOFS

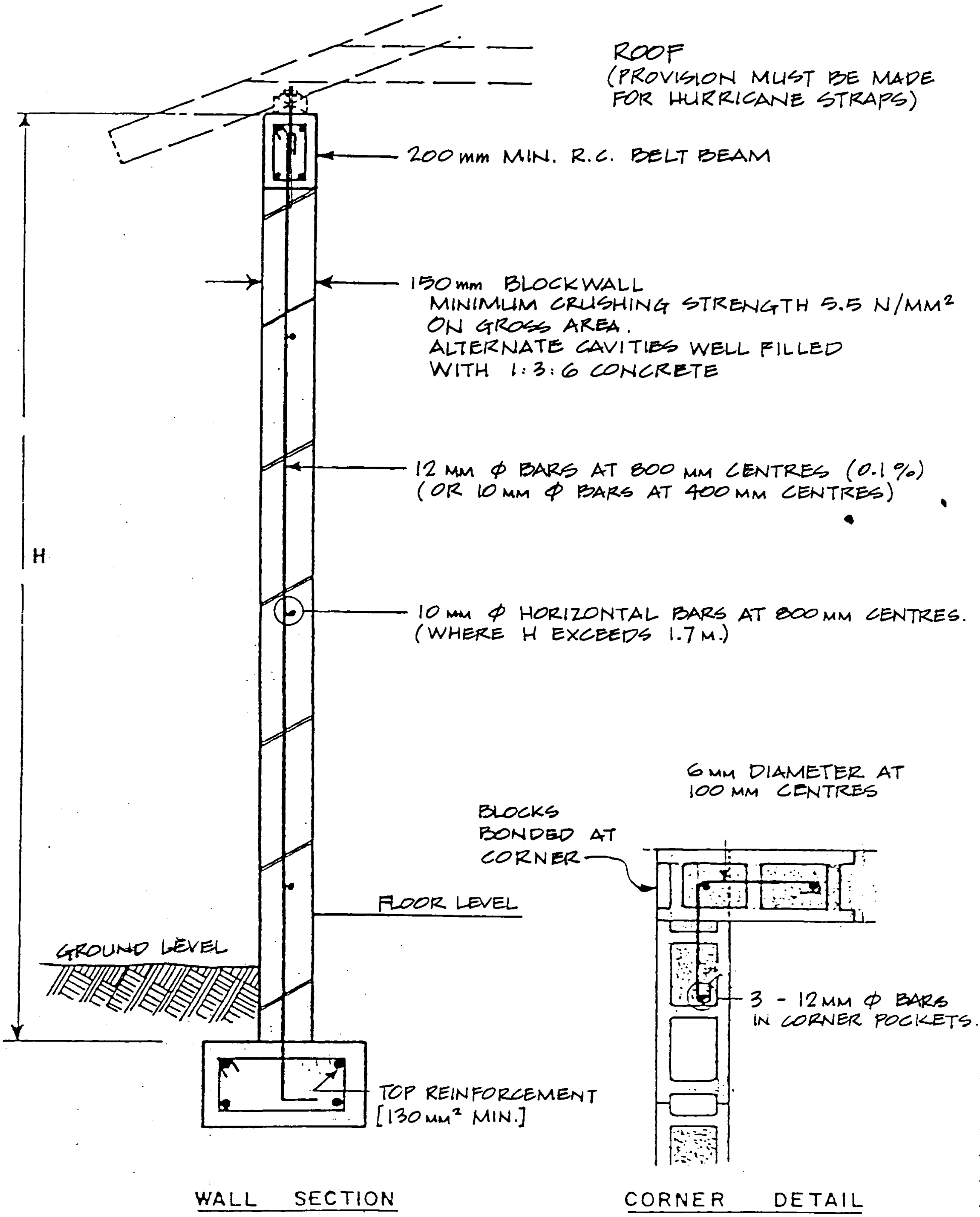


FIGURE 2.453.1

PROPOSED MINIMUM REQUIREMENTS WHERE DESIGN INDICATES NOMINAL REINFORCING.

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 4
BLOCK MASONRY

APPENDIX 1
SPECIAL REQUIREMENTS FOR MASONRY
OTHER THAN HOLLOW UNIT MASONRY

A100.0 GENERAL

Masonry other than hollow unit masonry shall be designed generally in accordance with all the foregoing Sections 4A, 4B and 4C except for the special requirements set out in this Appendix.

For the special case of small one and two-storey buildings in SPC 'A' or small one-storey buildings in other SPC's this Appendix may be used without reference to other sections. A future Part 5 will be added to this Code to deal in greater detail with this category of buildings.

A101.0 MATERIALS

A101.1 Reference Documents

The following standards apply to masonry materials and to the testing thereof:

MATERIALS AND DESIGN	STANDARD DESIGNATION
Building and Facing Brick	
Clay and Shale	ASTM C62, C216, C652
Sand-Lime	ASTM C73
Method of Test	ASMT C67
Concrete Masonry Units	
Hollow Load-Bearing	ASTM C90
Solid Load-Bearing	ASTM C145
Hollow Non-Load-Bearing	ASTM C129
Brick	ASTM C55
Method of Test	ASTM C140
Structural Clay Tile	
For Walls - Load-Bearing	ASTM C34, C212
For Walls - Non-Bearing	ASTM C56
For Floors	ASTM C57
Cast Stone	ACI 704
Unburned Clay	Uniform Building Code USA Standard 24-15
Cement	
Blended Hydraulic Cement Portland Cement and Air-Entraining Portland	*ASTM C595

Cement	ASTM C150
Masonry Cement	ASTM C91
Portland Cement	
(Ordinary or Rapid Hardening)	*BS 12
Portland Blast furnace	BS 146
Jamaican Standard Specification	
for Portland Cement	JS32/1974

Lime

Quicklime	ASTM C5
Aggregates for Mortar	ASTM C144
Aggregates for Grout	ASTM C404
Hydrated Lime for Masonry Purposes	ASTM C207

Aggregates

Aggregates for Mortar	ASTM C144
Aggregates for Grout	ASTM C404
Aggregates from Natural Sources	
for Concrete	BS 882

Mortar

Other than Gypsum	ASTM C270
Aggregates for Mortar	ASTM C144
Field Tests for Mortar	

Grout

Aggregates for Grout	ASTM C404
Field Tests for Grout	

* ASTM - American Society for Testing and Materials
BS - British Standards
JS - Jamaican Standard - Jamaica Bureau of Standards
For local equivalent refer to local Bureau of Standards

A101.2 **Criteria for Masonry Units**

Masonry units shall be of a type, quality, and grade consistent with the applicable provisions and intent of the referenced documents considering:

(i) The intended usage such as structural or non-structural.

(ii) The surrounding environment, such as the presence of water, contact with the ground, exposure to the weather and/or enclosure within a building.

Type, quality, grade and any similar additional special requirements of this Appendix or Section 4A for masonry units, all as applicable, shall be indicated on the design documents.

A101.3 **Initial Rate of Absorption**

At the time of laying, burned clay units and sandlime units shall have a rate of absorption not exceeding 0.12 ml per square centimeter during a period of one minute. Test procedures shall be in accordance with ASTM C67-73. In the absorption test the surface of the unit shall be held 3 mm below the surface of the water. Water content shall be that of the units to be laid, i.e., the units shall not be dried.

A101.4 **Brick Masonry Unit Surfaces for Grouted Masonry**

Masonry units for reinforced and unreinforced grouted masonry shall have all surfaces to which grout is to be applied capable of adhering to grout with sufficient tenacity to resist a shearing stress of 0.7 N/mm^2 after curing 28 days. Tests, when required, shall conform to Section 4A.

A101.5 **Re-use of Masonry Units**

Masonry units may be re-used when clean, whole, and in conformance with the requirements of this Section and those of the applicable reference documents. Conformance must be established by tests of representative samples.

A101.6 **Cast Stone**

Every cast stone unit more than 460 mm in any dimension shall conform to the requirements for concrete in Section 4A.

A101.7 **Natural Stone**

Natural stone shall be sound, clean and in conformity with other provisions of the Section 4.

A101.8 **Shrinkage of Concrete Units**

Concrete masonry untis used for structural purposes shall have a maximum linear shrinkage of 0.065 percent from the saturated to the oven-dry condition.

A102.0 **TYPES OF CONSTRUCTION**

The types of masonry construction described in Clause A102.1 may be used for structural or non-structural purposes and the type of masonry construction in A102.2 may be used for non-structural purposes subject to requirements of Section 4A and 4B.

A102.1 **Unburned Clay Masonry**

Unburned clay masonry is that form of construction with unburned clay stabilized with emulsified asphalt. Such units shall not be used in any building more than one storey in height. All footing walls which support masonry of unburned clay units shall extend to an elevation not less than 150 mm above the adjacent ground at all points.

A102.2 **Stone Masonry**

Stone masonry is that form of construction made with natural or cast stone with all joints thoroughly filled. In ashlar masonry, bond stones uniformly distibuted shall be provided to the extent of not less than 10 percent of the area exposed faces.

Rubble stone masonry 600 mm or less in thickness shall have bond stones with a maximum spacing of 1 m vertically and 1 m horizontally, and if the masonry is of greater thickness than 600 mm, shall have one bond stone for each 0.6 m^2 of wall surface on both sides.

A102.3 **Solid Masonry**

Solid masonry shall be solid concrete or clay masonry units laid contiguously in mortar. The bonding of adjacent wythes in bearing and non-bearing walls shall conform to one of the following methods:

Headers:

The facing and backing shall be bonded so that not less than 4 percent of the exposed face area is composed of solid headers extending not less than 75 mm into the backing. The distance between adjacent full length headers shall not exceed 600 mm vertically or horizontally. Where backing consists of two or more wythes, the headers shall extend not less than 75 mm into the most distant wythe or the backing wytes shall be bonded together with separate headers whose area and spacing conform to this clause.

Metal Ties:

The facing and backing shall be bonded with corrosion-resistant unit metal ties or cross wires or approved joint reinforcement for cavity walls. Unit ties shall be of sufficient length to engage all wythes, with ends embedded not less than 25 mm in mortar, or shall consist of two lengths, the inner embedded ends of which are hooked and lapped not less than 50 mm.

Where the space between metal tied wythes is solidly filled with mortar the allowable stresses and other provisions for masonry bonded walls shall apply.

A102.4 **Grouted Masonry - Multi-wythe Walls**

Grouted masonry is that form of construction made with brick or solid concrete units in which interior joints of masonry are filled by pouring grout therein as the work progresses. Only Type M or Type S mortar shall be used. When reinforced in accordance with Section 4A, masonry shall be classified as reinforced grouted masonry.

Grouting procedures for the space between wythes shall conform to the requirements given in sub-section 2.409. Coarse grout may be used in grout spaces 50 mm or more in width. Coarse grout shall be used where the least dimension of the grout space exceeds 125 mm.

TABLE A102-1
MAXIMUM RATIO OF HEIGHT TO THICKNESS AND MINIMUM THICKNESS OF MASONRY WALLS

Type of Masonry	Maximum ratio of unsupported height or length to thickness (1),(6)	Minimum nominal thickness (mm)(3)
STRUCTURAL WALLS:		
Unburned Clay Masonry	10	400
Stone Masonry	14	400
Hollow Unit Masonry (See Section 4A)	20	200
Solid Masonry	20	200
Grouted Masonry	20 (2)	150
Reinforced Grouted Masonry (See Section 4A)	25 (2)	150
Reinforced Hollow Unit Masonry (See Section 4A)	25 (2)	150
NON-STRUCTURAL AND PARTITIONS:(4)	30	100

(1) For cantilever walls, the actual height or length, as applicable, used to compute the actual thickness ratio shall be doubled.
(2) The minimum thickness requirements of Clause A102.0 shall also be satisfied.
(3) The thickness of plaster coatings may be considered in satisfying thickness ratios and minimum thickness requirements but shall not be used to take stresses.
(4) The maximum ratio of height or length may be increased and the minimum thickness may be decreased when justified by substantiating data.
(5) Nominal 100 mm loadbearing reinforced hollow clay unit masonry walls with a maximum unsupported height or length to thickness of 27 may be permitted provided net area unit strength exceeds 5.6 N/mm^2, units are laid in running bond, bar sizes do not exceed 12 mm with no more than two bars or one splice in a cell and joints are flushed cut, concave or a protruding v-section. Minimum bar covering where exposed to weather may be 40 mm.
(6) These thicknesses may be reduced to 150 mm for grouted walls and 200 mm for solid masonry walls in one-storey buildings when the wall is not over 2.75 m in total height, provided that when gable construction is used an additional 1.8 m in height is permitted to the peak of the gable.

TABLE A102-2
ALLOWABLE WORKING STRESSES IN UNREINFORCED MASONRY

N/mm^2

Mortar Type

Material (6)	M	S	M or S				N		
	Compression	Compression	Shear or Tension in Flexure (2,3)		Tension in Flexure (3,4,8)		Compression (1)	Shear or Tension in Flexural (2,3,8)	
Special Inspection required	No	No	Yes	No	Yes	No	No	Yes	No
Solid Brick Masonry									
>31.6 N/mm^2 (7)	1.75	1.57	0.14	0.07	0.28	0.14	1.40	0.10	0.05
17.6 - 31.5 N/mm^2 (7)	1.22	1.12	0.14	0.07	0.28	0.14	0.98	0.10	0.05
10.5 - 17.5 N/mm^2 (7)	0.87	0.80	0.14	0.07	0.28	0.14	0.70	0.10	0.05
Solid Concrete Masonry									
Grade N	1.22	1.12	0.08	0.04	0.17	0.08		0.08	0.04
Grade S	0.87	0.80	0.08	0.04	0.17	0.08		0.08	0.04
Grouted Masonry									
>31.6 N/mm^2 (7)	2.45	1.92	0.17	0.09	0.35	0.17			
17.6 - 31.5 N/mm^2 (7)	1.92	1.50	0.17	0.09	0.35	0.17			
10.5 - 17.5 N/mm^2 (7)	1.57	1.22	0.17	0.09	0.35	0.17			
Hollow Unit Masonry (5)	1.19	1.05	0.08	0.04	0.17	0.08	0.98	0.07	0.03
Cavity Wall Masonry									
Solid Units (5)									
>17.5 N/mm^2	0.98	0.91	0.08	0.04	0.21	0.10	0.77	0.07	0.03
10.5 - 17.5 N/mm^2	0.70	0.63	0.08	0.04	0.21	0.10	0.56	0.07	0.03
Hollow Units	0.50	0.42	0.08	0.04	0.21	0.10	0.35	0.07	0.03
Stone Masonry									
Cast Stone	2.80	2.52	0.06	0.03			2.24	0.06	0.03
Natural Stone	0.98	0.84	0.06	0.03			0.70	0.06	0.03
Unburned Clay Masonry	0.21	0.21	0.06	0.03					

1 Allowable axial or flexural compressive stresses in N/mm^2 on gross cross-sectional area (except as noted). The allowable working stresses in bearing directly under concentrated loads may be 50 percent greater than these values. Allowable axial stresses are only applicable if the maximum thickness ratios of Table A102-1 are not exceeded. Reduce these values by 20 percent when designing columns.

2 This value of tension is based on tension across a bed joint, i.e., vertically in the normal masonry work.

3 No tension allowed in stacked bond across head joints.

4 The values shown here are for tension in masonry in the direction of the bond, i.e. horizontally between supports.

5 Net bedded area or net cross-sectional area, whichever is more critical.

6 Strengths listed in this column are those of masonry units.

7 When the required strengths of the units exceed 17.5 N/mm^2, compression tests of the units conforming to the applicable reference documents shall be made. This shall not be required if certification acceptable to the Regulatory Agency accompany the units.

8 Allowable shear and tension stresses where lightweight concrete units are used are limited to 85 percent of the tabulated values.

TABLE A102-3
ASSUMED COMPRESSIVE STRENGTH OF MASONRY

f'_m

Type of Unit	Compressive Strength of Units, N/mm^2 or Grade	(N/mm^2) Type N Mortar	Type S Mortar	Type M Mortar
Solid Clay	98.0 N/mm^2 gross [1]	30.1[2,5]	37.1[2,5]	44.1[2,5]
and Net Area	84.0 N/mm^2 gross [1]	26.6[2,5]	32.2[2,5]	38.5[2,5]
of Hollow Clay	70.0 N/mm^2 gross [1]	23.1[2,5]	28.0[2,5]	32.2[2,5]
	56.0 N/mm^2 gross [1]	18.9[2,5]	23.1[2,5]	26.6[2,5]
	42.0 N/mm^2 gross [1]	15.4[5]	18.2[5]	21.0[5]
	28.0 N/mm^2 gross [1]	11.2	13.3	15.4[5]
	14.0 N/mm^2 gross [1]	7.7	8.4	9.1
Solid Concrete	42.0 N/mm^2 gross	9.5	16.8	16.8
and Net Areas	28.0 N/mm^2 gross	8.8	14.0	14.0
of Hollow	17.5 N/mm^2 gross	7.7	10.9	10.9
Concrete	10.5 N/mm^2 gross	6.1	8.1	8.1
	7.0 N/mm^2 gross	4.9	6.3	6.3

1 When the required strength of the units exceeds 21.0 N/mm^2 compression tests of the units conforming to the application reference documents shall be made.

2 When the assumed f'_m exceeds 18.2 N/mm^2 prism tests conforming to clause 2.410.9 shall be provided during construction. Certification of the units is not acceptable in lieu of tests.

3 Intermediate values may be interpolated.

4 Where grouted construction is used, the value of f'_m shall not exceed the compressive strength of the grout unless prism tests are provided during construction.

A103.0 MINIMUM REQUIREMENTS FOR WALLS OF BRICK OR STONE MASONRY

The following provisions are largely based on work by the Tropical Divsion of the Building Research Station U.K.

A103.1 Lateral Supports - Slenderness - Openings

(a) Every wall shall be supportd at right angles to the wall face by means of other walls, piers or buttresses.

(b) The distances between such lateral supports for walls shall not exceed:

(i) for loadbearing walls - 20 times the walls thickness for bricks, 14 times for stone.
(ii) for non-loadbearing walls - 30 times the wall thickness.

For the purposes of this clause A103.1 the wall thickness shall be taken to be the thickness of the wall excluding any surface finish or rendering.

(c) In any unframed building other than a garage:

(i) the total width of openings in any wall shall not exceed one half the length of that wall
(ii) the total width of openings in the external walls of any storey shall not exceed one-third of the length of the external walls of that storey
(iii) the horizontal distance between openings in a wall shall be not less than:

(a) twice the thickness of that wall or

(b) one third the width of the wider opening or

(c) 500mm

whichever is the greater

(iv) The horizontal distance bean opening in a load bearing wall and any corner of a wall or wall intersection shall not be less than :

(a) Twice the thickness of the wall or

(b) 500mm

whichever is the greater.

A103.2 Walls in Minor Seismic Areas With Average To Good Ground (Or Seismic Performance Category A)

(A) General Requirements

Single Storey Walls:

External walls and other load bearing walls not more than 3m high measured to the underside of the belt-beam required in Clause A103.6 shall be constructed of either:

Stone:

(i) Masonry set in mortar, and not less than 400 mm thick or

(ii) Burnt clay bricks set in mortar and not less than 210mm thick; or

Two Storey Walls:

External walls and other load bearing walls two storeys high shall have the upper storey constructed to comply with requirements for walls not exceeding 2.75m high as set out above. The lower storey walls shall be constructed of:

(i) Masonry set in mortar and not less than 450 mm thick; or

(ii) Burnt clay bricks set in mortar and not less than 210mm thick.

(B) Construction Details - Stone Masonry

(i) The part of the building on which the wall stands is roughened and cleaned to provide a good bond;

(ii) The stones are set in mortar not weaker than mix 1:4 cement to sand.

(iii) All joints are completely filled with mortar and all mortar is used within 25 minutes of water being added to dry mix;

(iv) There is not less than one through stone to every $0.6m^2$ of wall;

(v) All stones are wetted before being laid;

(vi) All walls are plumb and all walls, piers, corners and intersections are correctly bonded;

(C) Construction Details - Brick Walls

(i) The bricks are well burnt, sound, square edged and correct to shape and size within a tolerance of ± 6mm

(ii) The blocks or bricks are set in mortar not weaker than mix and all joints are completely filled with mortar; the mortar is used within 25 minutes of water being added to the dry mix

(iii) Expanded metal or purpose made reinforcement extending the whole width of the wall less 25mm cover on each side

(iv) Is tied or otherwise securely fixed to the vertical reinforcement at each end.

(v) Has an overlap of not less than 300 mm at laps.

A103.3 **Walls In Moderate Seismic Areas With Average To Good Ground (Or Seismic Performance Category B)**

(A) General Requirements

External walls and other load bearing walls not more than 3 m high measured to the underside of the belt-beam in Section 4A sub-section 2.414.

(i) Be of masonry set in mortar and not less stone than 400 mm thick or

(ii) Be of burnt clay bricks set in mortar and not less than 210 mm thick or

(iii) Have every wall at every corner or wall intersection suitable vertical reinforcement connecting the ring-beam with the foundation or solid floor; and

(iv) Have suitable horizontal reinforcement in courses not more than 450 mm apart: this reinforcement shall tie the walling to the vertical reinforcement provided to comply with sub-clause (iii) above

(B) Construction Details - Stone Masonry

(i) The part of the building on which the wall stands is roughened and cleaned to provide a good bond;

(ii) The stones are set in mortar not weaker than 1:4 cement :sand

(iii) All joints are completely filled with mortar and all mortar is used within 25 minutes of water being added to the dry mix;

(iv) There is not less than one through stone to every $0.6m^2$ of wall;

(v) All stones are wetted before being laid;

(vi) All walls are plumb and intersections are correctly bonded;

(vii) The horizontal reinforcement for a load bearing wall:

(a) Consists of three bars each 6 mm diameter set one in the middle of the wall and one about 50 mm in from each side of the wall; or

(b) Expanded metal or purpose made reinforcement extending the whole of the wall less 25 mm cover on each side.

(c) In any case is tied or otherwise securely fixed to the vertical reinforcement at each end; and

(d) Has an overlap of not less than 300 mm

The horizontal reinforcement for a wall which carries no load other than its own weight complies with B (vii) of these clauses except that only one bar is required set in the middle of the wall.

(C) <u>Construction Details - Brick Walls</u>

(1) The bricks are well burnt, sound, square edged and correct to shape and size within a tolerance of 6 mm

(2) The bricks are set in mortar not weaker than mix 1:4 cement and sand and all joints are completely filled with mortar; the mortar is used within 25 minutes of water being added to the dry mix.

(3) The part of the building on which the wall stands is roughened and cleaned to provide a good bond and all blocks and bricks are wetted before being laid;

(4) All walls are plumb and all walls, piers, corners and intersections are correctly bonded.

(5) The vertical reinforcement at each corner or wall intersection consists of two bars each at least 12 mm diameter and bent through a right angle at top and bottom and welded or tied to:

(a) The reinforcement in the belt-beam and

(b) The reinforcement in the foundation

(6) The horizontal reinforcement

(a) Consists of two bars each 6 mm diameter each set 25 mm in form a side of the wall; or

(b) Expanded metal or purpose made reinforcement extending the whole width of the wall less 25 mm cover on each side.

(c) Has an overlap of not less than 300 mm at laps.

A103.4 Walls In Major Seismic Areas With Average To Good Ground (Seismic Performance Category C)

(A) General Requirements

External walls and other load bearing walls not more than 3 m high measured to the underside of the beam-belt required in Clause A103.6 shall:

(i) Be of burnt clay bricks set in mortar and not less than 210 mm thick.

(B) Construction Details - Brick Walls

(i) The bricks are well burnt, sound, square edged and correct to shape and size within a tolerance of 6 mm.

(ii) The blocks or bricks are set in mortar not weaker than mix 1:4 cement to sand and all joints are completely filled with mortar: the mortar is used within 25 minutes of water being added to the dry mix.

(iii) The part of the building on which the wall stands is roughened and cleaned to provide a good bond and all blocks and bricks are wetted before being laid.

(iv) All walls are plumb and all walls, piers, corners and intersections are correctly bonded.

(v) The vertical reinforcement at corners, wall intersections and the sides of doors or windows consists of two bars each at least 12 mm diameter and bent through a right angle at top and bottom and welded or tied to:

(a) The reinforcement in the belt-beam, and

(b) The reinforcement in the lintel, and

(c) The reinforcement in the foundation

(vi) The horizontal reinforcement -

(a) Consists of two bars each 6 mm diameter each set 25 mm in from a side of the wall or

(b) Expanded metal or purpose made reinforcement extending the whole width of the wall less 25 mm cover on each side.

(c) Is tied or otherwise securely fixed to the vertical reinforcement at each end.

(d) Has an overlap of not less than 300 mm at joints.

A103.6 Belt Beams

(A) General Requirements

(i) This Regulation shall apply to all walls constructed of stone masonry or bricks other than infilling walls or curtain walls in framed buildings.

(ii) There shall be a suitable continuously reinforced concrete belt-beam on top of each wall.

(iii) There shall be another belt-beam at the level of each floor which is suspended above ground level; provided that where the floor is of reinforced concrete the belt-beam may be designed as part of the floor.

(iv) Each belt-beam shall be suitably connected to the walls or any other parts of the building on which it rests or which rests on it.

(v) Each belt-beam shall extend over the full width of the wall.

(B) Belt Beams in Minor Seismic Areas With Average To Good Ground (SPC "A")

(i) The beam is at least 150 mm deep.

(ii) It is reinforced with four bars each 12 mm in diameter held together with stirrups not more than 380 mm apart.

(iii) Each beam is fixed to any part of the building on which it rests or which rests on it by dowels; these dowels penetrate at least 100 mm into the beam to the start of the bend 230 mm into the other part of the building, and are made of 12 mm diameter bars.

(C) <u>Belt Beams In Moderate Seismic Areas With Average To Good Ground (SPC "B")</u>

(i) The beam is at least 200 mm deep

(ii) It is reinforced with four bars each 12 mm diameter held together with stirrups not more than 250 mm apart.

(iii) Each beam is fixed to any part of the building on which it rests or which rests on it by dowels: these dowels penetrate at least 100 mm into the beam to the start of the bend 230 mm into the other part of the building, and are made of 12 mm diameter bar set not more than 1.3 m apart measured along the walls.

(D) <u>Belt Beams In Major Seismic Areas With Average To Good Ground (SPC "C")</u>

(i) The beam is at least 200 mm deep

(ii) It is reinforced with four bars each 16 mm diameter held together with stirrups not more than 250 mm apart.

(iii) Each beam is fixed to any part of the building on which it rests or which rests on it by dowels: these dowels penetrate at least 100 mm into the beam to the point where the bar bends and 230 mm into the other part of the building, and are made of 12 mm diameter bars set not more than 1.3 m apart measured along the walls.

PART 2
STRUCTURAL DESIGN REQUIRMENTS

SECTION 4
BLOCK MASONRY

APPENDIX 2
SPECIAL INSPECTIONS AND TESTS

SPECIAL INSPECTIONS AND TESTS

The following special inspections and tests are required to qualify for higher allowable stresses listed in Section 4A Table 2.416.1 and for buildings in seismic performance category D.

(A) Special Inspection

Special inspection shall be provided as follows:

For the examination of materials and/or certification of materials for compliance.

For the observation of measurement and mixing of field-mixed mortar and grout including checks on consistency.

For the determination of the moisture conditions of the masonry units at the time of laying.

For periodic observations of the laying of masonry units with special attention to joints including preparations prior to buttering, portions to be filled, shoving, etc.

For observation of the bonding of units in the walls between wythes and at corners and intersections.

For the proper placement of reinforcement including splices, clearances, and support.

For observation of the construction of chases, recesses, and the placement of pipes, conduits, and other weakening elements.

For inspection of grout spaces immediately prior to grouting including the removal of mortar fins as required, removal of dirt and debris, and the conditions at the bottom of the grout space. For high lift work this shall be done prior to the closing of cleanouts and shall also include the proper sealing of cleanouts.

For preparation, or supervision of preparation, of required samples such as mortar, grout, and prisms.

For the observation of grout placement with special attention to procedures to obtain filling of required spaces, the avoidance of segregation, and proper consolidiation and reconsolidation.

(B) Tests and/or Certifications

Tests and/or certifications shall be performed and/or supplied as follows:

For mortar, grout, and prisms. One prism test series shall be made for each 460 square metres of wall. Alternatively, a series of both mortar and grout tests shall be made on the first three consecutive days of the week and on each third day thereafter.

In addition, when f_m' is equal to or greater than 18.2 N/mm^2 or when f_m' is to be established by tests, a minimum of three prism test series shall be made during the progress of the work. When f_m' is to be established by tests there shall be an initial prism test series prior to the start of construction.

The requirements for numbers of test series apply separately for each variation of type of masonry construction except for the total number for a building.

For masonry units. When shipments of masonry units are not identified and accompanied by certification, one series of tests for strength, absorption, saturation, moisture content, shrinkage, and modulus of rupture shall be made for each 460 square metres of wall or equivalent. When the reference document or standard for the units has no acceptance or rejection limits for a test, the tests need not be made.

Grouted masonry, seismic performance Category D. One series of core tests for shear bond shall be made for each 480 square metres of wall or equivalent.

For cement used for mortar and grout, certification acceptable to the Regulatory Agency shall accompany the cement when the required volume of cement exceeds 500 sacks.

For reinforcement. One tensile and bend test shall be made for each 2 1/2 tons of fraction thereof of each size of reinforcing. Testing is not required if the reinforcement is identified by heat number and is accompanied with a certified report of the mill analysis.

For other tests, performance shall be as indicated in Section 4A and the relevant material standards.

Where the number of tests series is not defined, one test or test series, as applicable, shall be made for each 480 square metres of wall or equivalent.

Load Tests

When a load test is required the member or portion of the structure under consideration shall be subject to a superimposed load equal to twice the specified live load plus 1/2 of the dead load. This load shall be left in position for a period of 24 hours before removal. If, during the test or upon removal of the load, the member or portion of the structure shows evidence of failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or where lawful, a lower rating shall be established. A flexural member shall be considered to have passed the test if the maximum deflection "D" at the end of 24 hours period neither exceeds:

$$D = \frac{L}{200}$$

nor

$$D = \frac{L^2}{4000\ t}$$

and the beams and slabs show a recovery of at least 75 percent of the observed deflection within 24 hours after removal of the load where:

L = span of the member in feet
t = thickness or depth of the member in feet.

Test Criteria

Masonry prisms, mortar and grout samples, and masonry cores shall be prepared and tested in accordance with the procedures in this Section 4.

PART 2
STRUCTURAL DESIGN REQUIRMENTS

SECTION 4
BLOCK MASONRY

APPENDIX 3
LIST OF REFERENCES

LIST OF REFERENCES

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FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

$Feet^2$ (ft^2) x 0.0929030	= $Meters^2$ (m^2)
Acres x 4046.86	= $Meters^2$ (m^2)
$Miles^2$ (mi^2) x 2.58999	= $Kilometers^2$ (km^2)

VOLUME

$Feet^3$ (ft^3) x 0.02831685	= $Meters^3$ (m^3)
$Feet^3$ (ft^3) x 28.31685	= Liters**
$Yards^3$ (yd^3) x 0.764555	= $Meters^3$ (m^3)
Acre feet x 1233.48	= $Meters^3$ (m^3)
Gallons (gal) x 0.00454609	= $Meters^3$ (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

$Feet^3$ per second (ft^3/s) x 0.02831685	= $Meters^3$ per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= $Meters^3$ per second (m^3/s)
Acre-feet per day x 0.0142764	= $Meters^3$ per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= $Meters^3$ per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 ($lb\text{-}s/ft^2$) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

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* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

NOTES

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 6

Structural Design Requirements
REINFORCED AND PRESTRESSED CONCRETE

1985

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 6
REINFORCED AND PRE-STRESSED CONCRETE

Caribbean Community Secretariat
Georgetown
Guyana

1985

PART 2

SECTION 6

TABLE OF CONTENTS

FOREWORD

The objective of drafting this section 2.600 of the Caribbean Uniform Building Code is to provide the minimum requirements for the design construction and material properties of reinforced and prestressed concrete.

ACI-318-M (1983) was chosen as the "Base Code" for section 2.600 of the Caribbean Uniform Building Code. The main reasons for the selection of ACI-318 were as follows:

(i) The provision of recommendations of the design of earthquake resistant structures (outlined in Appendix A of ACI-318) was the most significant factor influencing the choice of ACI-318 as the Base Code.

(ii) While BS CP110-1972 is used today in conjunction with the "SEAOC" Code it is generally felt that the compatibility of this combination is questionable. The "SEAOC" Code is cross-referenced to ACI-318.

The "Base code" ACI-318 has been augmented to include the following provisions in section 2.600.

(i) Servicibility requirements for the calculation of long-term deflections and calculation of crack-widths section, 9.5/9.6.

(ii) General provision of cantilevered shear walls and coupled shear walls, Appendix A section A.10 (BASED ON NZS 3101 and NZS 4203).

(iii) Detailing recommendations for Fire-Resistance to RC/Pre-stressed Beams, Floors, Slabs, Columns and Walls have been included in Appendix E. These recommendations are based on BS CP110-1972.

In addition, recommendations for the use of structural concrete (that is provisions regarding the use of aggregates in the Caricom Countries of Barbados, Dominica, Grenada, Guyana, St. Lucia, St. Vincent and Trinidad and Tobago have been included in Appendix F. Recommendations regarding the use of aggregates in other Caricom Countries not mentioned here will be included as data becomes available.

Finally all reinforcement used in section 2.600 shall be in accordance with any of the following standards BS 4449, BS 4461 or ASTM 706.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

The number and digit corresponding to the Part and Section in the Part (2.6) have been omitted. The numbers that remain correspond to the sub-sections and articles. Therefore, the number 3.2 corresponds to Part 2, Section 6, sub-section 3, Article 2 (2.603.2).

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

PART 2

STRUCTURAL DESIGN REQUIREMENTS

SECTION 6A

REINFORCED AND PRE-STRESSED CONCRETE

GENERAL

PART 2

SECTION 6 A

SUB-SECTION 1 - GENERAL REQUIREMENTS

1.1 Scope

1.1.1 This Code provides minimum requirements for the design and construction of reinforced concrete structural elements of any structure erected under the requirements of the General Building Code of which this Code forms a part.

1.1.2 This Code supplements the General Building Code and shall govern in all matters pertaining to design and construction wherever this Code is in conflict with requirements in the General Building Code.

1.1.3 This Code shall govern in all matters pertaining to design, construction and material properties whenever this Code is in conflict with requirements contained in other specifications referenced in this Code.

1.1.4 For special structures, such as arches, tanks, reservoirs, bins and silos, blast-resistant structures, and chimneys, provisions of this Code shall govern where applicable.

1.1.5 This Code does not govern the design and installation of portions of concrete piles and drilled piers embedded in ground.

1.1.6 Use of plain concrete for structural members shall be in accordance with ACI standard "Building Code Requirements for Structural Plain Concrete (ACI 318.1M)"

1.1.7 SPECIAL PROVISIONS FOR EARTHQUAKE RESISTANCE

1.1.7.1 In regions of low seismic risk, provisions of Appendix A shall not apply.

1.1.7.2 In regions of moderate or high seismic risk, provisions of Appendix A shall be satisfied. See Article A.2.1.

1.1.7.3 The seismic risk level of a region shall be regulated by the general Building Code of which this Code forms a part, or determined by Local Authority.

1.2 Permits and Drawings

1.2.1 Copies of design drawings, typical details, and specifications for all reinforced concrete constructions shall bear the seal of a registered engineer or architect and shall be filed with the Building Department as a permanent record before a permit to construct such work will be issued. These drawings, details, and specifications shall show:

(a) Name and date of issue of Code and supplement to which design conforms

(b) Live load and other loads used in design

(c) Specified compressive strength of concrete at stated ages or stages of construction for which each part of structure is designed

(d) Specified strength or grade of reinforcement

(e) Size and location of all structural elements and reinforcement

(f) Provision for dimensional changes resulting from creep, shrinkage, and temperature

(g) Magnitude and location of pre-stressing forces

(h) Type and location of splices of reinforcement

1.2.2 Calculations pertinent to design shall be filed with the drawings when required by the Building Official. When automatic data processing is used, design assumptions and identified input and output data may be submitted in lieu of calculations. Calculations may be supplemented by model analysis.

1.2.3 Building Official means the officer or other designated authority charged with the administration and enforcement of this Code, or his duly authorized representative.

1.3 Inspection

1.3.1 Concrete construction shall be inspected throughout the various work stages by a competent engineer or architect, or by a competent representative responsible to that engineer or architect.

1.3.2 The Inspector shall require compliance with design drawings and specifications and keep a record that shall cover:

(a) Quality and proportions of concrete materials
(b) Construction and removal of forms, re-shoring
(c) Placing of reinforcement
(d) Mixing, placing and curing of concrete

(e) Sequence of erection and connection of precast members
(f) Tensioning of pre-stressing tendons
(g) Any significant construction loading on completed floors, members, or walls
(h) General progress of work.

1.3.3 When the ambient temperature falls below $5^{o}C$ or rises above $35^{o}C$, a complete record shall be kept of concrete temperatures and of protection given to concrete during placement and curing.

1.3.4 Records of inspection required in Articles 1.3.2 and 1.3.3 shall be kept available to the Building Official during progress of work and for 2 years after completion of the project and shall be preserved by inspecting engineer or architect for that purpose.

1.4 Approval of Special Systems of Design or Construction

Sponsors of any system of design or construciton within the scope of this Code, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by this Code, shall have the right to present the data so submitted , to require tests, and to formulate rules governing design and construction of such systems to meet the intent of this Code. When approved by the Building Official and promulgated (these rules) shall be of the same force and effect as the provisions of this Code.

SUB-SECTION 2 - DEFINITIONS

2.1 The following terms are defined for general use in this Code. Specialized definitions appear in individual chapters.

Admixture — Material other than water, aggregate, or hydraulic cement, used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties.

Aggregate — Granular material, such as sand, gravel crushed stone, and iron blast-furnace slag, used with a cementing medium to form a hydraulic-cement concrete or mortar.

Aggregate, low-density — Aggregate with a dry, loose weight of 1100 kg/m^3 or less.

Anchorage — In post-tensioning, a device used to anchor tendon to concrete member; in pre-tensioning, a device used to anchor tendon during hardening of concrete.

Bonded tendon	Pre-stressing tendon that is bonded to concrete either directly or through grouting.
Building Official	See Article 1.2.3
Column	Member with a ratio of height-to-least-lateral dimension of 3 or greater used primarily to support axial compressive load.
Composite concrete flexural members	Concrete flexural members of precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to loads as a unit.
Concrete	Mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate and water, with or without admixtures.
Concrete, speci-ified compressive strength of, (f'_c)	Compressive strength of concrete used in design and evaluated in accordance with provisions of Sub-section 4, expressed in megapascals (MPa). Whenever the quantity f'_c is under a radical sign, square root of numerical value only is intended, and result has units of megapascals (MPa).
Concrete, struc-tural low-density	Concrete containing low-density aggregate that conforms to Article 3.3 and has an air-dry unit mass as determined by "Test Method for Unit Weight of Structural Lightweight Concrete" (ASTM C 567), not exceeding 1900 kg/m^3. In this Code, a low-density concrete without natural sand is termed "all-low-density concrete" and low-density concrete in which all of the fine aggregate consists of normal density sand is termed "sand-low density concrete".
Curvature friction	Friction resulting from bends or curves in the specified pre-stressing tendon profile.
Deformed rein-forcement	Deformed reinforcing bars, bar mats, deformed wire, welded smooth wire fabric, and welded deformed wire fabric conforming to Article 3.5.3
Development length	Length of embedded reinforcement required to develop the design strength of reinforcement at a critical section. See Article 9.3.3
Effective depth of section (d)	Distance measured from extreme compression fiber to centroid of tension reinforcement.
Effective pre-stress	Stress remaining in pre-stressing tendons after all losses have occurred, excluding effects of dead load and superimposed load.

Embedment length	Length of embedded reinforcement provided beyond a critical section.
Jacking force	In pre-stressed concrete, temporary force exerted by device that introduces tension into pre-stressing tendons.
Load, dead	Dead weight supported by a member, as defined by General Building Code of which this Code forms a part (without load factors).
Load, factored	Load, multiplied by appropriate load factors, used to proportion members by the strength design method of this Code. See Articles 8.1.1 and 9.2.
Load, live	Live load specified by General Building Code of which this Code forms a part (without load factors).
Load, service	Load specified by General Building Code of which this Code forms a part (without load factors).
Modulus of of elasticity	Ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material. See Article 8.5
Pedestal	Upright compression member with a ratio of unsupported height to average least lateral dimension of less than 3.
Plain concrete	Concrete that does not conform to definition of reinforced concrete.
Plain reinforcement	Reinforcement that does not conform to definition of deformed reinforcement. See Article 3.5.4
Post-tensioning	Method of pre-stressing in which tendons are tensioned after concrete has hardened.
Precast concrete	Plain or reinforced concrete element cast elsewhere than its final position in the structure.
Pre-stressed concrete	Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.
Pretensioning	Method of pre-stressing in which tendons are tensioned before concrete is placed.
Reinforced concrete	Concrete reinforced with no less than the minimum amount required by this Code, pre-stressed or non-pre-stressed, and designed on the assumption that the two materials act together in resisting forces.

Reinforcement	Material that conforms to Article 3.5, excluding pre-stressing tendons unless specifically included.
Span length	See Article 8.7
Spiral reinforcement	Continuously wound reinforcement in the form of a cylindrical helix.
Splitting tensile strength (f_{ct})	Tensile strength of concrete detemined in accordance with "Specification for Lightweight Aggregates for Structural Concrete" (ASTM C 330). See Article 4.1.4.
Stirrup	Reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wore fabric (smooth or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term "ties" to those in compression members). See also Tie.
Strength, design	Nominal strength multiplied by a strength reduction factor ϕ. See Article 9.3.
Strength, nominal	Strength of a member or cross-section calculated in accordance with provisions and assumptions of the strength design method of this Code before application of any strength reduction factors. See Article 9.3.1.
Strength, required	Strength of a member or cross-section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in this Code. See Article 9.1.1.
Stress	Intensity of force per unit area.
Tendon	Steel element such as wire, cable, bar, rod, or strand, or a bundle of such elements, used to impart prestress to concrete.
Tie	Loop of reinforcing bar or wire enclosing longitudinal reinforcement. See also "stirrup".
Transfer	Act of transferring stress in pre-stressing tendons from jacks or pretensioning bed to concrete member.
Wall	Member, usually vertical, used to enclose or separate spaces.

Wobble friction	In pre-stressed concrete, friction caused by unintended deviation of pre-stressing sheath or duct from its specified profile.
Yield strength	Specified minimum yield strength or yield point of reinforcement in megapascals. Yield strength or yield point shall be determined in tension according to applicable ASTM specifications as modified by Article 3.5 of this Code.

PART 2

STRUCTURAL DESIGN REQUIREMENTS

SECTION 6B

REINFORCED AND PRE-STRESSED CONCRETE

STANDARDS FOR TESTS AND MATERIALS

PART 2
SECTION 6 B

SUB-SECTION 3 - MATERIALS

3.1 **Tests of Materials**

3.1.1 Building Official shall have the right to order testing of any materials used in concrete construction to determine if materials are of quality specified.

3.1.2 Tests of materials and of concrete shall be made in accordance with standards of the American Society for Testing and Materials, listed in Article 3.8.1.

3.1.3 A complete record of tests of materials and of concrete shall be available for inspection during progress of work and for 2 years after completion of the project, and shall be preserved by inspecting engineer or architect for that purpose.

3.2 **Cements**

3.2.1 Cement shall conform to one of the following specifications for portland cement:

(a) "Specification for Portland Cement" (ASTM C 150).

(b) "Specification for Blended Hydraulic Cements" (ASTM C 595), excluding Types S and SA which are not intended as principal cementing constituents of structural concrete.

3.2.2 Cement used in the work shall correspond to that on which selection of concrete proportions was based. See Article 4.2.

3.3 **Aggregates**

3.3.1 Concrete aggregates shall conform to one of the following specifications:

(a) "Specification for Concrete Aggregates" (ASTM C 33)
(b) "Specification for Lightweight Aggregates for Structural Concrete" (ASTM C 330)

3.3.2 Aggregates failing to meet the specifications listed in Article 3.3.1, but which have been shown by special test or actual service to produce concrete of adequate strength and durability may be used where authorized by the Building Official.

3.3.3 Nominal maximum size of coarse aggregate shall be not larger than:

(a) 1/5 the narrowest dimension between sides of forms, nor
(b) 1/3 the depth of slabs, nor
(c) 3/4 the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, or pre-stressing tendons or ducts.

These limitations may be waived if, in the judgment of the Engineer, workability and methods of consolidation are such that concrete can be placed without honeycomb or voids.

3.4 **Water**

3.4.1 Water used in mixing concrete shall be clean and free fron injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be deleterious to concrete or reinforcement.

3.4.2 Mixing water for pre-stressed concrete or for concrete that will contain aluminum embedments, including that portion of mixing water contributed in the form of free moisture on aggregates, shall not contain deleterious amounts of chloride ion. See Article 4.5.4.

3.4.3 Non-potable water shall not be used in concrete unless the following are satisfied:

3.4.3.1 Selection of concrete proportions shall be based on concrete mixes using water from the same source.

3.4.3.2 Mortar test cubes made with non-potable mixing water shall have 7-day and 28-day strengths equal to at least 90 percent of strengths of similar specimens made with potable water. Strength test comparison shall be made on mortars, identical except for the mixing water, prepared and tested in accordance with "Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-inch or 50-mm Cube Specimens)" (ASTM C 109).

3.5 Metal Reinforcement

3.5.1 Reinforcement shall be deformed reinforcement, except that plain reinforcement may be used for spirals or tendons; and reinforcement consisting of structural steel, steel pipe, or steel tubing may be used as specified in this Code.

3.5.2 Reinforcing bars to be welded shall be indicated on the drawings and welding procedure to be used shall be specified. ASTM reinforcing bar specifications, except for ASTM A706, shall be supplemented to require a report of material properties necessary to conform to welding procedures specified in "Structural Welding Code - Reinforcing Steel" (AWS D1.4) of the American Welding Society.

3.5.3 DEFORMED REINFORCEMENT

3.5.3.1 Deformed reinforcing bars shall conform to the following specifications:

"Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement" (ASTM A 706).

3.5.3.2 Deformed reinforcing bars with a specified yield strength f_y exceeding 400 MPa may be used, provided f_y shall be the stress corresponding to a strain of 0.35 percent and the bars otherwise conform to one of the ASTM specifications listed in Article 3.5.3.1. See Article 9.4

3.5.3.3 Bar mats for concrete reinforcement shall conform to "Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement" (ASTM A 184). Reinforcing bars used in bar mats shall conform to one of the specifications listed in Article 3.5.3.1

3.5.3.4 Deformed wire for concrete reinforcement shall conform to "Specification for Deformed Steel Wire for Concrete Reinforcement" (ASTM A 496), except that wire shall not be smaller than size D4 and for wire with a specified yield strength f_y exceeding 400 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent.

3.5.3.5 Welded smooth wire fabric for concrete reinforcement shall conform to "Specification for Welded Steel Wire Fabric for Concrete Reinforcement" (ASTM A 185), except that for wire with a specified yield strength f_y exceeding 400 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent. Welded intersections shall not be spaced farther apart than 310 mm in direction of calculated stress, except for wire fabric used as stirrups in accordance with Article 12.13.2.

3.5.3.6 Welded deformed wire fabric to concrete reinforcement shall conform to "Specification for Welded Deformed Steel Wire Fabric for Concrete Reinforcement" (ASTM A 497), except that for wire with a specified yield strength f_y exceeding 400 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent. Welded intersections shall not be spaced farther apart than 400 mm in direction of calculated stress, except for wire fabric used as stirrups in accordance with Article 12.13.2.

3.5.3.7 Reinforcing bars may be galvanized or epoxy coated in accordance with "Specification for Zinc Coated (Galvanized) Steel Bars for Concrete Reinforcement" (ASTM A 767) or "Specification for Epoxy Coated Reinforcing Steel Bars" (ASTM A 775). Zinc or epoxy-coated reinforcing bars shall conform to one of the specifications listed in Article 3.5.3.1.

3.5.4 PLAIN REINFORCEMENT

3.5.4.1 Plain bars for spiral reinforcement shall conform to the specification listed in Article 3.5.3.1 (a), (b) or (c).

3.5.4.2 Smooth wire for spiral reinforcement shall conform to "Specification for Cold-Drawn Steel Wire for Concrete Reinforcement" (ASTM A 82), except that for wire with a specified yield strength f_y exceeding 400 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent.

3.5.5 PRE-STRESSING TENDONS

3.5.5.1 Tendons for pre-stressed reinforcement shall conform to one of the following specifications:

(a) Wire conforming to "Specifications for Uncoated Stress-Relieved Steel Wire for Pre-stressed Concrete" (ASTM A 421).

(b) Low-relaxation wire conforming to "Specification for Uncoated Stress-Relieved Steel Wire for Pre-stressed Concrete" including Supplement "Low-Relaxation Wire" (ASTM A 421).

(c) Strand conforming to "Specification for Uncoated Seven-Wire Stress-Relieved Steel Strand for Pre-stressed Concrete" (ASTM A 416).

(d) Low-relaxation strand conforming to "Specification for Uncoated Seven-Wire Stress-Relieved Steel Strand for Pre-stressed Concrete" including Supplement "Low-Relaxation Strands" (ASTM A 416).

(e) Bar conforming to "Specification for Uncoated High-Strength Steel Bar for Pre-stressed Concrete" (ASTM A 722).

3.5.5.2 Wire, strands, and bars not specifically listed in ASTM A 421, A 416, or A 722 may be used provided they conform to minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed in ASTM A 421, A 416, or A 722.

3.5.6 STRUCTURAL STEEL, STEEL PIPE, OR TUBING.

3.5.6.1 Structural steel used with reinforcing bars in composite compression members meeting requirements of Article 10.14.7 or 10.14.8 shall conform to one of the following specifications:

(a) "Specification for Structural Steel" (ASTM A 36).
(b) "Specification for High-Strength Low-Alloy Structural Steel" (ASTM A 242).
(c) "Specification for High-Strength Low-Alloy Structural Manganese Vanadium Steel" (ASTM A 441).
(d) "Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality" (ASTM A 572).
(e) "Specification for High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick" (ASTM A 588).

3.5.6.2 Steel pipe or tubing for composite compression members composed of a steel encased concrete core meeting requirements of Article 10.14.6 shall conform to one of the following specifications:

(a) Grade B of "Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless" (ASTM A 53).

(b) "Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes" (ASTM A 500).

(c) "Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing" (ASTM A 501).

3.6 Admixtures

3.6.1 Admixtures to be used in concrete shall be subject to prior approval by the Engineer.

3.6.2 An admixture shall be shown capable of maintaining essentially the same composition and performance throughout the work as the product used in establishing concrete proportions in accordance with Article 4.2.

3.6.3 Calcium chloride or admixtures containing chloride from impurities other than from admixture ingredients shall not be used in pre-stressed concrete, in concrete containing embedded aluminum, or in concrete cast against stay-in-place galvanized metal forms. See Articles 4.5.3.1 and 4.5.4.

3.6.4 Air-entraining admixtures shall conform to "Specification for Air-Entraining Admixtures for Concrete" (ASTM C 260).

3.6.5 Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and accelerating admixtures shall conform to "Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete" (ASTM C 618).

3.7 Storage of materials.

3.7.1 Cement and aggregates shall be stored in such manner as to prevent deterioration or intrusion of foreign matter.

3.7.2 Any material that has deteriorated or has been contaminated shall not be used for concrete.

3.8 Standards cited in this Code.

3.8.1 Standards fo the American Society for Testing and Materials referred to in this Code are listed below with their serial designations, including year of adoption or revision, and are declared to be part of this Code as if fully set forth herein:

A36-81a	Standard Specification for Structural Steel
A53-82	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
A82-79	Standard Specification for Cold-Drawn Steel Wire for Concrete Reinformcement
A184-79	Standard Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement

A185-79	Standard Specification for Welded Steel Wire Fabric for Concrete Reinforcement
A242-81	Standard Specification for High-Strength Low-Alloy Structural Steel
A416-80	Standard Specification for Uncoated Seven-Wire Stress-Relieved Steel Strand for Pre-stressed Concrete
A421-80	Standard Specification for Uncoated Stress-Relieved Steel Wire for Pre-stressed Concrete
A441-81	Standard Specification for High-Strength Low-Alloy Structural Manganese Vanadium Steel
A496-78	Standard Specification for Welded Deformed Steel Wire Fabric for Concrete Reinforcement
A497-79	Standard Specification for Welded Deformed Steel Wire Fabric for Concrete Reinforcement
A500-82a	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A501-81	Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
A572-82	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
A588-82	Standard Specidication for High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick
A615M-82 (S1)	Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement (Metric) including Supplementary Requirements (S1)
A616-82a	Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
A617-82a	Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
A706-82a	Standard Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement

A722-75 (1981)	Standard Specification for Uncoated High-Strength Steel Bar for Pre-stressing Concrete
A767-79	Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A775-81	Standard Specification for Epoxy-Coated Reinforcing Steel Bars
C31-69 (1980)	Standard Method of Making and Curing Concrete Test Specimens in the Field
C33-82	Standard Specification for Concrete Aggregates
C39-81	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C42-77	Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C94-81	Standard Specification for Ready-Mixed Concrete
C109-80	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-inch or 500 mm Cube Specimens)
C144-81	Standard Specification for Aggregate for Masonry Mortar
C150-81	Standard Specification for Portland Cement
C172-82	Standard Method of Sampling Freshly Mixed Concrete
C192-81	Standard Method of Making and Curing Concrete Test Specimens in the Laboratory
C260-77	Standard Specification for Air-Entraining Admixtures for Concrete
C330-80	Standard Specification for Lightweight Aggregates for Structural Concrete
C494-81	Standard Specification for Chemical Admixtures for Concrete
C496-71 (1979)	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C567-80	Standard Test Method for Unit Weight of Structural Lightweight Concrete

C595-82	Standard Specification for Blended Hydraulic Cements
C618-80	Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
C685-81	Standard Specification for Concrete made by Volumetric Batching and Continuous Mixing

3.8.2 "Structural Welding Code - Reinforcing Steel" (AWS D1.4-79) of the American Welding Society is declared to be part of this Code as if fully set forth herein.

3.8.3 "Building Code Requirements for Structural Plain Concrete (ACI 318.1M-83)" is declared to be part of this Code as if fully set forth herein.

PART 2

STRUCTURAL DESIGN REQUIREMENTS

SECTION 6C

REINFORCED AND PRE-STRESSED CONCRETE

CONSTRUCTION REQUIREMENTS

SUB-SECTION 4 - CONCRETE QUALITY

4.1 General

4.1.1 Concrete shall be proportioned to provide an average compressive strength as prescribed in Article 4.3.2. Concrete shall be produced to minimize frequency of strengths below f'_c as prescribed in Article 4.7.2.3.

4.1.2 Requirements for f'_c shall be based on tests of cylinders made and tested as prescribed in Article 4.7.2.

4.1.3 Unless otherwise specified, f'_c shall be based on 28-day tests. If other than 28 days, test age for f'_c shall be as indicated in design drawings or specifications.

4.1.4 Where design criteria in Articles 9.5.2.3, 11.2 and 12.2.3.3 provide for use of a splitting tensile strength value of concrete, laboratory tests shall be made in accordance with "Specification for Light weight Aggregates for Structural Concrete" (ASTM C330) to establish value of f_{ct} corresponding to specified value of f'_c.

4.1.5 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

4.2 Selection of Concrete Proportions

4.2.1 Proportions of materials for concrete shall be established to provide:

(a) Workability and consistency to permit concrete to be worked readily into forms and around reinforcement under conditions of placement to be employed, without segregation or excessive bleeding.

(b) Resistance to special exposures as required by Article 4.5.

(c) Conformance with strength test requirements of Article 4.7.

4.2.2 Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

4.2.3 Concrete proportions, including water-cement ratio, shall be established on the basis of field experience and/or trial mixtures with materials to be employed (Article 4.3), except as permitted in Article 4.4 or required by Article 4.5.

4.3 Proportioning on the Basis of Field Experience and/or Trial Mixtures

4.3.1 STANDARD DEVIATION

4.3.1.1 Where a concrete production facility has test records, a standard deviation shall be established. Test records from which a standard deviation is calculated:

(a) shall represent materials, quality control procedures, and conditions similar to those expected, and changes in materials and proportions within the test records shall not have been more restricted than those for proposed work.

(b) shall represent concrete produced to meet a specified strength or strengths f'_c within 7 MPa of that specified for proposed work.

(c) shall consist of at least 30 consecutive tests or two groups of consecutive tests totalling at least 30 tests as defined in Article 4.7.1.4, except as provided in Article 4.3.1.2.

4.3.1.2 Where a concrete production facilty does not have test records meeting requirements of Article 4.3.1.1, but does have a record based on 15 to 29 consecutive tests, a standard deviation may be established as the product of the calculated standard deviation and modification factor of Table 4.3.1.2. To be acceptable, the test record must meet requirements (a) and (b) of Article 4.3.1.1, and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

TABLE 4.3.1.2

MODIFICATION FACTOR FOR STANDARD DEVIATION WHEN LESS THAN 30 TESTS ARE AVAILABLE

No. of tests*		Modification factor for standard deviation**
less than	15	Use table 4.3.2.2
	15	1.16
	20	1.08
	25	1.03
	30 or more	1.00

* Interpolate for intermediate numbers of tests

** Modified standard deviation to be used to determine required average strength f'_{cr} from Article 4.3.2.1

4.3.2 REQUIRED AVERAGE STRENGTH

4.3.2.1 Required average compressive strength f'_{cr} used as the basis for selection of concrete propositions shall be the larger of Eq. (4-1) or (4-2) using a standard deviation calculated in accordance with Article 4.3.1.1 or Article 4.3.1.2.

$$f'_{cr} = f'_c + 1.34s \qquad (4\text{-}1)$$

or

$$f'_{cr} = f'_c + 2.33s - 3.5 \qquad (4\text{-}1)$$

4.3.2.2 When a concrete production facility does not have field strength test records for calculation of standard deviation meeting requirements of Article 4.3.1.1 or Article 4.3.1.1, required average strength f'_{cr} shall be determined from Table 4.3.2.2 and documentation of average strength shall be in accordance with requirements of Article 4.3.3.

TABLE 4.3.2.2

REQUIRED AVERAGE COMPRESSIVE STRENGTH WHEN DATA ARE NOT AVAILABLE TO ESTABLISH A STANDARD DEVIATION

Specified compressive strength f'_c MPa	Required average compressive strength f'_{cr} MPa
Less than 20 MPa	$f'_c + 7.0$
20 - 35	$f'_c + 8.5$
Over 35	$f'_c + 10.0$

4.3.3 DOCUMENTATION OF AVERAGE STRENGTH

Documentation that proposed concrete proportions will produce an average compressive strength equal to or greater than required average compressive strength (Article 4.3.2) may consist of a field strength test record, several strength test records, or trial mixtures.

4.3.3.1 When test records are used to demonstrate that proposed concrete proportions will produce the required average strength f'_{cr} (Article 4.3.2), such records shall represent materials and conditions similar to those expected. Changes in materials, conditions, and proportions within the test records shall not have been more restricted than those for proposed work. For the purpose of documenting average strength potential, test records consisting of less than 30 but not less than 10 consecutive tests may be used provided test records encompass a period of time not less than 45 days. Required concrete proportions may be established by interpolation between the strengths and proportions of two or more test records each of which meets other requirements of this section.

4.3.3.2 When an acceptable record of field test results is not available, concrete proportions may be established based on trial mixtures meeting the following restrictions.

(a) Combination of materials shall be those for proposed work.

(b) Trial mixtures having proportions and consistencies required for proposed work shall be made using at least three different water-cement ratios or cement contents that will produce a range of strengths encompassing the required average strength f'_{cr}.

(c) Trial mixtures shall be designed to produce a slump within ± 20 mm of maximum permitted, and for air-entrained concrete, within ± 0.5 percent of maximum allowable air content.

(d) For each water-cement ratio or cement content, at least three test cylinders for each test age shall be made and cured in accordance with "Method of Making and Curing Concrete Test Specimens in the Laboratory" (ASTM C 192). Cylinders shall be tested at 28 days or at test age designated for determination of f'_c.

(e) From results of cylinder tests a curve shall be plotted showing relationship between water-cement ratio or cement content and compressive strength at designated test age.

(f) Maximum water-cement ratio or minimum cement content for concrete to be used in proposed work shall be that shown by the curve to produce the average strength required by Article 4.3.2, unless a lower water-cement ratio of higher strength is required by Article 4.5.

4.4 Proportioning by water-cement ratio

4.4.1 If data required by Article 4.3 are not available, permission may be granted to base concrete proportions on water-cement ratio limits in Table 4.4.

TABLE 4.4

MAXIMUM PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE WHEN STRENGTH DATA FROM FIELD EXPERIENCE OR TRIAL MIXTURES ARE NOT AVAILABLE

Specified compressive strength f'_c MPa*	Absolute water-cement ratio by mass	
	Non-air-entrained concrete	Air-entrained concrete
17	0.66	0.54
20	0.60	0.49
25	0.50	0.39
30	0.40	**
35	**	**

* 28-day strength. With most materials, water-cement ratios shown will provide average strengths greater than indicated in Article 4.3.2 as being required.

** For strengths above 30 MPa (non-air-entrained concrete) and 35 MPa (air-entrained concrete), concrete proportions shall be established by methods of Article 4.3.

4.4.2 Table 4.4 shall be used only for concrete to be made with cements meeting strength requirements for Types I, IA, II, IIA, III, IIIA, or V of "Speculation for Portland Cement" (ASTM C 150), or Types IS, IS-A, IS(MS), IS-A(MS), I(SM), I(SM)-A, IP, IP-A, I(PM), I(PM)-A, IP(MS), IP-A(MS), or P of "Speculation for Blended Hydraulic Cements" (ASTM C 595), and shall not be applied to concrete containing low-density aggregates or admixtures other than those for entraining air.

4.4.3 Concrete proportioned according to water-cement ratio limits prescribed in Table 4.4 shall also conform to the special exposure requirements of Article 4.5 and to the compressive strength test criteria of Article 4.7.

4.5 Special exposure requirements

4.5.1 Normal density and low-density concrete exposed to freezing and thawing or de-icer chemicals shall be air entrained with air content indicated in Table 4.5.1. Tolerance on air content as delivered shall be $\pm$ 1.5 percent. For specified compressive strength f'_c greater than 35 MPa, air content indicated in Table 4.5.1 may be reduced 1 percent.

TABLE 4.5.1

TOTAL AIR CONTENT FOR FROST-RESISTANT CONCRETE

Nominal maximum aggregate size mm*	Air content, percent	
	Severe exposure	Moderate exposure
9.5	7 1/2	6
12.5	7	5 1/2
19.0	6	5
25.0	6	4 1/2
37.5	5 1/2	4 1/2
50**	5	4
75**	4 1/2	3 1/2

* See ASTM C 33 for tolerances on oversize for various nominal maximum size designations.

** These air contents apply to total mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 37.5 mm is removed by handpicking or sieving and air content is determined on the minus 37.5 mm fraction of mix. (Tolerance on air content as delivered applies to this value). Air content of total mix is computed from value determined on the minus 37.5 mm fraction.

TABLE 4.5.2

REQUIREMENTS FOR SPECIAL EXPOSURE CONDITIONS

Exposure condition	Maximum water-cement ratio, normal density aggregate concrete	Minimum f'_c low-density aggregate concrete, MPa
Concrete intended to be water-tight:		
(a) Concrete exposed to fresh water	0.50	25
(b) Concrete exposed to brackish water or seawater	0.45	30
Concrete exposed to freezing and thawing in a moist condition:		
(a) Curbs, gutters, guardrails or thin sections	0.45	30
(b) Other elements	0.50	25
(c) In presence of de-icing chemicals	0.45	30
For corrosion protection for reinforced concrete exposed to de-icing salts, brackish water, seawater or spray from these sources	0.40*	33

* If minimum concrete cover required by Article 7.7 is increased by 10 mm, water-cement ratio may be increased to 0.45 for normal density concrete, or f'_c reduced to 30 MPa for low-density concrete.

4.5.2 Concrete that is intended to be watertight or concrete that will be subject to freezing and thawing in a moist condition shall conform to requirements of Table 4.5.2.

4.5.3 Concrete to be exposed to sulfate-containing solutions shall conform to requirements of Table 4.5.3 or be made with a cement that provides sulfate resistance and used in concrete with maximum water-cement ratio or minimum compressive strength from Table 4.5.3.

4.5.3.1 Calcium chloride as an admixture shall not be used in concrete to be exposed to severe or very severe sulfate-containing solutions, as defined in Table 4.5.3

TABLE 4.5.3
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate exposure	Water soluble sulfate (SO_4) in soil percent by mass	Sulfate (SO_4) in water, ppm	Cement type	Normal density aggregate concrete: Maximum water-cement ratio, by mass*	Low density aggregate concrete: Minimum compressive f'_c MPa*
Negli.	0.00-0.10	0-150	--	--	--
Mod.**	0.10-0.20	150-1500	II, IP(MS), IS(MS)	0.50	25
Severe	0.20-2.00	1500-10000	V	0.45	30
Very severe	Over 2.00	Over 10000	V plus pozzolan***	0.45	30

* A lower water-cement ratio of higher strength may be required for watertightness or for protection against corrosion of embedded items or freezing and thawing (Table 4.5.2)

** Seawater

*** Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

4.5.4 For corrosion protection, maximum water soluble chloride ion concentrations in hardened concrete at an age of 28 days contributed from the ingredients including water, aggregates, cementitious materials and admixtures shall not exceed limits of Table 4.5.4.

TABLE 4.5.4
MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION

Type of member	Maximum water soluble chloride ion (Cl^-) in concrete, percent by mass of cement
Pre-stressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

4.5.5 When reinforced concrete will be exposed to de-icing salts, brackish water, seawater, or spray from these sources, requirements of Table 4.5.2 for water-cement ratio or concrete strength and minimum concrete cover requirements of Article 7.7 shall be satisfied.

4.6 Average strength reduction

As data become available during construction, amount by which value of f'_{cr} must exceed specified value of f'_c may be reduced, provided:

(a) 30 or more test results are available and the average of test results exceeds that required by Article 4.3.2.1 using a standard deviation calculated in accordance with Article 4.3.1.1, or

(b) 15 to 29 test resuls are available and the average of test results exceeds that required by Article 4.3.2.1 using a standard deviation calculated in accordance with Article 4.3.1.2, and

(c) the special exposure requirements of Article 4.5 are met.

4.7 Evaluation and acceptance of concrete

4.7.1 FREQUENCY OF TESTING

4.7.1.1 Samples for strength tests of each class of concrete placed each day shall be taken not less that once a day, nor less than once for each 120 m^3 of concrete, nor less than once for each 500 m^2 of surface area for slabs or walls.

4.7.1.2 On a given project, if total volume of concrete is such that frequency of testing required by Article 4.7.1.1 would provide less than five strength tests for a given class of concrete, tests shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

4.7.1.3 When the total quantity of a given class of concrete is less than 40 m^3, strength tests may be waived by the Building Official, if in his judgment evidence of satisfactory strength is provided.

4.7.1.4 A strength test shall be the average of the strengths of two cylinders made from the same sample of concrete and tested at 28 days or at a test age designated for determination of f'_c.

4.7.2 LABORATORY-CURED SPECIMENS

4.7.2.1 Samples for strength tests shall be taken in accordance with "Method of Sampling Freshly Mixed Concrete" (ASTM C 172).

4.7.2.2 Cylinders for strength tests shall be moulded and laboratory-cured in accordance with "Method of Making and Curing Concrete Test Specimens in the Field" (ASTM C 31) and tested in accordance with "Test Method for Compressive Strength of Cylindrical Concrete Specimens" (ASTM C 39).

4.7.2.3 Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

(a) Average of all sets of three consecutive strength tests equal or exceed f'_c.

(b) No individual strength test (average of two cylinders) falls below f'_c by more than 3.5 MPa.

4.7.2.4 If either of the requirements of Article 4.7.2.3 are not met, steps shall be taken to increase the average of subsequent strength test results. Requirements of Article 4.7.4 shall be observed if requirement of Article 4.7.2.3(b) is not met.

4.7.3 FIELD-CURED SPECIMENS

4.7.3.1 The Building Official may require strength tests of cylinders cured under field conditions to check adequacy of curing and protection of concrete in the structure.

4.7.3.2 Field-cured cylinders shall be cured under field conditions in accordance with Article 7.4 of "Method of Making and Curing Concrete Test Specimens in the Field" (ASTM C 31).

4.7.3.3 Field-cured test cylinders shall be moulded at the same time and from the same samples as laboratory-cured test cylinders.

4.7.3.4 Procedures for the protecting and curing concrete shall be improved when strength of field-cured cylinders at test age designated for determination of f'_c is less than 85 percent of that of companion laboratory-cured cylinders. The 85 percent may be waived if field-cured strength exceeds f'_c by more than 3.5 MPa.

4.7.4 INVESTIGATION OF LOW-STRENGTH TEST RESULTS

4.7.4.1 If any strength test (Article 4.7.1.4) of laboratory-cured cylinders falls below specified value of f'_c by more than 3.5 MPa (Article 4.7.2.3(b)) or if tests of field-cured cylinders indicate deficiencies in protection and curing (Article 4.7.3.4), steps shall be taken to assure that load-carrying capacity of the structure is not jeopardized.

4.7.4.2 If the likelihood of low-strength concrete is confirmed and computations indicate that load-carrying capacity may have been significantly reduced, tests of cores drilled from the area in question may be required in accordance with "Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" (ASTM C 42). In such case, three cores shall be taken for each strength test more than 3.5 MPa below specified value of f'_c.

4.7.4.3 If concrete in the structure will be dry under service conditons, cores shall be air dried (temperture 15 to 30 C, relative humidity less than 60 percent) for 7 days before test and shall be tested dry. If concrete in the structure will be more than superficially wet under service conditions, cores shall be immersed in water for at least 40 hr and be tested wet.

4.7.4.4 Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f'_c and if no single core is less than 75 percent of f'_c. To check testing accuracy, locations represented by erratic core strengths may be retested.

4.7.4.5 If criteria of Article 4.7.4.4 are not met, and if structural adequacy remains in doubt, the responsible authority may order load tests as outlined in sub-section 20 for the questionable portion of the structure, or take other appropriate action.

SUB-SECTION 5 MIXING AND PLACING CONCRETE

5.1 Preparation of equipment and place of deposit

5.1.1 Preparation before concrete placement shall include the following:

(a) All equipment for mixing and transporting concrete shall be clean.
(b) All debris and ice shall be removed from spaces to be occupied by concrete.
(c) Forms shall be properly coated.
(d) Masonry filler units that will be in contact with concrete shall be well drenched.
(e) Reinforcement shall be thoroughly clean of ice or other deleterious coatings.
(f) Water shall be removed from place of deposit before concrete is placed unless a tremie is to be used or unless otherwise permitted by the Building Official.

(g) All laitance and other unsound material shall be removed before additonal concrete is placed against hardened concrete.

5.2 Mixing

5.2.1 All concrete shall be mixed until there is a uniform distribution of materials and shall be discharged completely before mixer is recharged.

5.2.2 Ready-mixed concrete shall be mixed and delivered in accordance with requirements of "Specification for Ready-Mixed Concrete" (ASTM C 94) or "Specification for Concrete Made by Volumetric Batching and Continuous Mixing" (ASTM C 685).

5.2.3 Job-mixed concrete shall be mixed in accordance with the following:

(a) Mixing shall be done in a batch mixer of approved type.

(b) Mixer shall be rotated at a speed recommended by the manufacturer.

(c) Mixing shall be continued for at least 1.5 min after all materials are in the drum, unless a shorter time is shown to be satisfactory by the mixing uniformity tests of "Specification for Ready-Mixed Concrete" (ASTM C 94).

(d) Materials handling, batching, and mixing shall conform to applicable provisions of "Specification for Ready-Mixed Concrete" (ASTM C 94).

(e) A detailed record shall be kep to identify:

(1) number of batches produced;
(2) proportions of materials used;
(3) approximate location of final deposit in structure;
(4) time and date of mixing and placing.

5.3 Conveying

5.3.1 Concrete shall be conveyed from mixer to place of final deposit by methods that will prevent separation or loss of materials.

5.3.2 Conveying equipment shall be capable of providing a supply of concrete at site of placement without separation of ingredients and without interruptions sufficient to permit loss of plasticity between successive increments.

5.4 Depositing

5.4.1 Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing.

5.4.2 Concreting shall be carried on at such a rate that concrete is at all times plastic and flows readily into spaces between reinforcement.

5.4.3 Concrete that has partially hardened or been contaminated by foreign materials shall not be deposited in the structure.

5.4.4 Re-tempered concrete or concrete that has been remixed after initial set shall not be used unless approved by the Engineer.

5.4.5 After concreting is started, it shall be carried on as a continuous operation until placing of a panel or section, as defined by its boundaries or predetermined joints, is completed except as permitted or prohibited by Article 6.4.

5.4.6 Top surfaces of vertically formed lifts shall be generally level.

5.4.7 When construction joints are required, joints shall be made in accordance with Article 6.4.

5.4.8 All concrete shall be thoroughly consolidated by suitable means during placement and shall be thoroughly worked around reinforcement and embedded fixtures and into corners of forms.

5.5 Curing

5.5.1 Concrete (other than high-early-strength) shall be maintained above 10 C and in a moist condition for at least the first 7 days after placement, except when cured in accordance with Article 5.5.3.

5.5.2 High-early-strength concrete shall be maintained above 10 C and in a moist condition for at least the first 3 days, except when cured in accordance with Article 5.5.3.

5.5.3 ACCELERATED CURING

5.5.3.1 Curing by high pressure steam, steam at atmospheric pressure, heat and moisture, or other accepted processes, may be employed to accelerate strength gain and reduce time of curing.

5.5.3.2 Accelerated curing shall provide a compressive strength of the concrete at the load stage considered at least equal to required design strength at that load stage.

5.5.3.3 Curing process shall be such as to produce concrete with a durability at least equivalent to the curing method of Article 5.5.1 or 5.5.2.

5.5.4 Supplementary strength tests in accordance with Article 4.7.3 may be required to assure that curing is satisfactory.

5.6 Cold Weather Requirements

5.6.1 Adequate equipment shall be provided for heating concrete materials and protecting concrete during freezing or near-freezing weather.

5.6.2 All concrete materials and all reinforcement, forms, fillers, and ground with which concrete is to come in contact shall be free from frost.

5.6.3 Frozen materials or materials containing ice shall not be used.

5.7 Hot weather requirements

During hot weather, proper attention shall be given to ingredients, production methods, handling, placing, protection, and curing to prevent excessive concrete temperatures of water evaporation that may impair required strength or serviceability of the member or structure.

SUB-SECTION 6 - FORMWORK, EMBEDDED PIPES, AND CONSTRUCTION JOINTS

6.1 Design of formwork

6.1.1 Forms shall result in a final structure that conforms to shapes, lines and dimensions of the members as required by the design drawings and specifications.

6.1.2 Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

6.1.3 Forms shall be properly braced or tied together to maintain position and shape.

6.1.4 Forms and their supports shall be designed so as not to maintain position and shape.

6.1.5 Design of formwork shall include consideration of the following factors:

(a) Rate and method of placing concrete

(b) Construction loads, including vertical, horizontal, and impact loads

(c) Special form requirements for construction of shells, folded plates, domes, architectural concrete, or similar types of elements.

6.1.6 Forms for pre-stressed concrete members shall be designed and constructed to permit movement of the member without damage during application of pre-stressing force.

6.2 Removal of forms and shores

6.2.1 No construction loads shall be supported on, nor any shoring removed from, any part of the structure under construction except when that portion of the structure in combination with remaining forming and shoring system has sufficient strength to safely support its weight and loads placed thereon.

6.2.1.1 Sufficient strength shall be demonstrated by structural analysis considering proposed loads, strength of forming and shoring system, and concrete strength data. Concrete strength data may be based on tests of field-cured cylinders or, when approved by the Building Official, on other procedures to evaluate concrete strength. Structural analysis and concrete strength test data shall be furnished to the Building Official when so required.

6.2.2 No construction loads exceeding the combination of superimposed dead load plus specified live load shall be supported on any unshored portion of the structure under construction, unless analysis indicates adequate strength to support such additional loads.

6.2.3 Forms shall be removed in such manner as not to impair safety and serviceability of the structure. All concrete to be exposed by form removal shall have sufficient strength not be to damaged thereby.

6.2.4 Form supports for pre-stressed concrete members may be removed when sufficient pre-stressing has been applied to enable pre-stressed members to carry their dead load and anticipated construction loads.

6.3 Conduits and Pipes Embedded in Concrete

6.3.1 Conduits, pipes and sleeves of any material not harmful to concrete and within limitations of Article 6.3 may be embedded in concrete with approval of the Engineer, provided they are not considered to replace structurally the displaced concrete.

6.3.2 Conduits and pipes of aluminum shall not be embedded in structural concrete unless effectively coated or covered to prevent aluminum-concrete reaction or electrolytic action between aluminum and steel.

6.3.3 Conduits, pipes and sleeves passing through a slab, wall or beam shall not significantly impair the strength of the construction.

6.3.4 Conduits and pipes, with their fittings, embedded within a column shall not displace more than 4 percent of the area of cross-section on which strength is calculated or which is required for fire protection.

6.3.5 Except when plans for conduits and pipes are approved by the Structural Engineer, conduits and pipes embedded within a slab, wall or beam (other than those merely passng through) shall satisfy the following:

6.3.5.1 They shall not be larger in outside dimension that 1/3 the overall thickness of slab, wall or beam in which they are embedded.

6.3.5.2 They shall not be spaced closer than 3 diameters or widths on center.

6.3.5.3 They shall not significantly impair the strength of the construction.

6.3.6 Conduits, pipes, and sleeves may be considered as structurally replacing in compression the displaced concrete provided:

6.3.6.1 They are not exposed to rusting or other deterioration.

6.3.6.2 They are of uncoated or galvanized iron or steel not thinner than standard Schedule 40 steel pipe.

6.3.6.3 They have a nominal inside diameter not over 50 mm and are spaced not less than 3 diameters on centers.

6.3.7 In addition to other requirements of Article 6.3, pipes that contain liquid, gas or vapor may be embedded in structural concrete under the following conditions:

6.3.7.1 Pipes and fittings shall be designated to resist effects of the material, pressure, and temperature to which they will be subjected.

6.3.7.2 Temperature of liquid, gas, or vapor shall not exceed 70^{o} C.

6.3.7.3 Maximum pressure to which any piping or fittings shall be subjected shall not exceed 1.5 MPa above atmospheric pressure.

6.3.7.4 All pipings and fittings except as provided in Article 6.3.7.5 shall be tested as a unit for leaks before concrete placement. Testing pressure above atmospspheric pressure shall be 50 percent in excess of pressure to which pipings and fittings may be subjected, but minimum testing pressure shall not be less than 1 MPa above atmospheric pressure. Pressure test shall be held for 4 hr with no drop in pressure except that which may be cuased by air temperature.

6.3.7.6 Drain pipes and other piping designed for pressures of not more than 10 kPa above atmospheric pressure need not be tested as required in Article 6.3.7.4.

6.3.7.6 Pipes carrying liquid, gas, or vapor that is explosive or injurious to health shall be tested again as specified in Article 6.3.7.4 after concrete has hardened.

6.3.7.7 No liquid, gas, or vapor except water not exceeding 30 C nor 0.3 MPa pressure, shall be placed in the pipes until the concrete has attained its design strength.

6.3.7.8 In solid slabs, piping, unless it is for radiant heating or snow melting, shall be placed between top and bottom reinforcement.

6.3.7.9 Concrete cover for pipes and fittings shall not be less than 40 mm for concrete exposed to earth or weather, or 20 mm for concrete not exposed to weather or in contact with ground.

6.3.7.10 Reinforcement with an area not less than 0.002 times area of concrete section shall be provided normal to piping.

6.3.7.11 Piping and fittings shall be assembled by welding, brazing, solder-sweating, or other equally satisfactory method. Screw connections shall not be permitted. Piping shall be so fabricated and installed that cutting, bending, or displacement of reinforcement from its proper location will not be required.

6.4 Construction Joints

6.4.1 Surface of concrete constructions joints shall be cleaned and laitance removed.

6.4.2 Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.

6.4.3 Constructions joints shall be so made and located as not to impair the strength of the structure. Provision shall be made for transfer of shear and other forces through construction joints. See Article 11.7.9.

6.4.4 Construction joints in floors shall be located within the middle third of spans of slabs, beams, and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams

6.4.5 Beams, girders, or slabs supported by columns or walls shall not be cast or erected until concrete in the verticl support members in no longer plastic.

6.4.6 Beams, girders, haunches, drop panels and capitals shall be placed momolithically as part of a slab system unless otherwise shown in design drawings or specifications.

SUB-SECTION 7 - DETAILS OF REINFORCEMENT

7.1 Standard Hooks

The term "Standard hook" is used in this Code shall mean one of the following:

7.1.1 180-deg bend plus $4d_b$ extension, but not less than 60 mm at free end of bar.

7.1.2 90-deg bend plus $12d_b$ extension, at free end of bar.

7.1.3 For stirrup and tie hooks*

(a) 15 bar and smaller,90-deg bend plus $6d_b$ extension at free end of bar, or

(b) 20 and 25 bar 90-deg bend plus $12d_b$ extension at free end of bar, or

(c) 25 bar and smaller, 135-deg bend plus $6d_b$ extension free end of bar.

* For closed ties defined as hoops in Appendix A, a 135-deg bend plus an extension of at least 10db (see section A.1.).

7.2 Minimum bend diameters

7.2.1 Diameter of bend measured on the inside of the bar, other than for stirrups and ties in sizes 10 through 15, shall not be less than the values in Table 7.2.

7.2.2 Inside diameter of bend for stirrups and ties shall not be less thn $4d_b$ for 15 bar and smaller, For bar larger than 15, diameter of bend shall be in accordance with Table 7.2.

7.2.3 Inside diameter of bend in welded wire fabric (smooth or deformed) for stirrups and ties shall not be less than $4d_b$ for deformed wire larger than D6 and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from nearest welded intersection.

TABLE 7.2
MINIMUM DIAMETERS OF BEND

Bar Size	Minimum Diameter
No. 10 through No. 25	$6d_b$
No. 30 and No. 35	$8d_b$
No. 45 and No. 55	$10d_b$

7.3 Bending

7.3.1 All reinforcement shall be bent cold, unless otherwise permitted by the Engineer.

7.3.2 Reinforcement partially embedded in concrete shall not be field bent, except as shown on the design drawings or permitted by the Engineer.

7.4 Surface conditions of reinforcement

7.4.1 At the time concrete is placed, metal reinforcement shall be free from mud, oil, or other non-metalic coatings that adversely effect bonding capacity.

7.4.2 Metal reinforcement, except pre-stressing tendons, with rust, mill scale, or a combination of both shall be conidered satisfatory, provided the minimum dimensions (including height of deformations) and weight of a hand-wire-brushed test specimen are not less than applicable ATSM specification requirements.

7.4.3 Pre-stressing tendons shall be clean and free of oil, dirt, scale, pitting and excessive rust. A light oxide is permissible.

7.5 **Placing reinforcement**

7.5.1 Reinforcement, pre-stressing tendons, and ducts shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in Article 7.5.2.

7.5.2 Unless otherwise specified by the Engineer, reinforcement, pre-stressing tendons, and pre-stressing ducts shall be placed ithin the following tolerances:

7.5.2.1 Tolerance for depth d, and minimum concrete cover in flexural members, walls and compression members shall be as follows:

	Tolerance on d	Tolerance on minimum concrete cover
$d \leq 200$ mm	±10 mm	-10 mm
$d > 200$ mm	±12 mm	-12 mm

Except that tolerance for the clear distance to formed soffits shall be minus 6 mm and tolerance for cover shall not exceed minus 1/3 the minimum concrete cover required in the design drawings or specifications.

7.5.2.2 Tolerance for longitudinal location of bends and ends of reinforcement shall be ±50 mm except at discontinuous ends of members where tolerance shall be ±12 mm.

7.5.3 Welded wire fabric (with wire size not greater that W5 or D5) used in slabs not exceeding 3 m in span may be curved from a point near the top of slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support.

7.5.4 Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the Engineer.

7.6 **Spacing limits for reinforcement**

7.6.1 Clear distance between parallel bars in a layer shall be not less than d_b nor 25 mm. See also Article 3.3.3.

7.6.2 Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above bars in the bottom layer with clear distance between layers not less than 25 mm.

7.6.3 In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall be not less than $1.5d_b$ nor 40 mm. See also Article 3.3.3.

7.6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

7.6.5 In walls and slabs other than concrete joist construction, primary flexural reinforcement shall be spaced not farther apart than three times the wall or slab thickness, nor 500 mm.

7.6.6 BUNDLED BARS

7.6.6.1 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.

7.6.6.2 Bundled bars shall be enclosed within stirrups or ties.

7.6.6.3 Bars larger than 35 shall not be bundled in beams.

7.6.6.4 Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least 40 d_b stagger.

7.6.6.5 Where spacing limitations and minimum concrete cover are based on bar diameter d_b, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent area.

7.6.7 PRE-STRESSING TENDONS AND DUCTS

7.6.7.1 Clear distance between pretensioning tendons at each end of a member shall be not less than $4d_b$ for wire, nor $3d_b$ for strands. See also Article 3.3.3. Closer vertical spacing and bundling to tendons may be permitted in the middle portion of the span.

7.6.7.2 Post-tendoning ducts may be bundled if shown that concrete can be satisfactorily placed and if provision is made to prevent the tendons, when tensioned, from breaking through the duct.

7.7 Concrete protection for reinforcement

7.7.1 CAST-IN-PLACE CONCRETE (NON-PRE-STRESSED)

The following minimum concrete cover shall be provided for reinforcement:

		Minimum cover, mm
(a)	Concrete cast against and permanently exposed to earth.	70
(b)	Concrete exposed to earth or weather:	
	No. 20 through No. 55 bars	50
	No. 15 bar, W31 or D31 wire, and smaller	40
(c)	Concrete not exposed to weather or in contact with ground:	
	Slabs, walls, joists:	
	No. 45 and No. 55	40
	No. 35 bar or smaller	20
	Beams, columns:	
	Primary reinforcement, ties stirrups, spirals	40
	Shells, folded plate members:	
	No. 20 bar and larger	20
	No. 15 bar, W31 or D31 wire, and smaller	15

7.7.2 PRECAST CONCRETE (MANUFACTURED UNDER PLANT CONTROL CONDITIONS)

The following minimum concrete cover shall be provided for reinforcement:

		Minimum cover, mm
(a)	Concrete exposed to earth or weather:	
	Wall panels:	
	No. 45 and No. 55 bars	40

No. 35 bar and smaller	20
Other members:	
No. 45 and No. 55 bars	50
No. 20 through No. 35 bars	40
No. 15 bar, W31 or D31 wire, and smaller	30
(b) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists:	
No. 45 and No. 55 bars	30
No. 35 bar and smaller	15
Beams, columns:	
Primary reinforcement	d_b but not less than 15 and need not exceed 40
Ties, stirrups, spirals	10
Shells, folded plate members:	
No. 20 bar and larger	15
No. 15 bar, W31 or D31 wire, and smaller	10

7.7.3 PRE-STRESSED CONCRETE

7.7.3.1 The following minimum concrete cover shall be provided for pre-stressed and non-pre-stressed reinforcement, ducts, and end fittings, except as provided in Articles 7.7.3.2 and 7.7.3.3:

	Minimum Cover, mm
(a) Concrete cast against and permanently exposed to earth	70
(b) Concrete exposed to earth or weather:	
Wall panels, slabs, joists	30
Other members	40
(c) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists	20
Beams, columns:	
Primary reinforcement	40
Ties, stirrups, spirals	20

Shells, folded plate members: 15 bar, W31 or D31 wire, and smaller	10
Other reinforcement	d_b but not less than 20

7.7.3.2 For pre-stressed concrete members exposed to earth, weather, or corrosive environments, and in which permissable tensile stress of Article 18.4.2(b) is exceeded, minimum cover shall be increased 50 percent.

7.7.3.3 For pre-stressed concrete members manufactured under plant control conditions, minimum concrete cover for non-pre-stressed reinforcement shall be as required in Article 7.7.2.

7.7.4 BUNDLED BARS

For bundled bars, minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 50 mm; except for concrete cast against and permanently exposed to earth, minimum cover shall be 70 mm.

7.7.5 CORROSIVE ENVIRONMENTS

In corrosive environments or other severe exposure conditions, the amount of concrete protection shall be suitably increased, and denseness and non-porosity of protecting concrete shal be considered, or other protection shall be provided.

7.7.6 FUTURE EXTENSIONS

Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

7.7.7 FIRE PROTECTION

When the General Building Code (of which this Code forms part) requires a thickness of cover for fire protection greater than the minimum concrete cover specified in Article 7.7, such greater thicknesses shall be used.

7.8 Special reinforcing details for columns

7.8.1 OFFSET BARS

Offset bent longitudinal bars shall conform to the following:

7.8.1.1 Slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.

7.8.1.2 Horizontal support at offset bends shall be provided by lateral ties, spirals, or parts of the floor construction. Horizontal support provided shall be designed to resist 1-1/2 times the horizontal component of the computed force in the inclined portion of an offset bar. Lateral ties or spirals, if used, shall be placed not more than 150 mm from points of bend.

7.8.1.3 Portions of bar above and below an offset shall be parallel to axis of column.

7.8.1.4 Offset bars shall be bent before placement in the forms See Article 7.3.

7.8.1.5 Where a column face is offset 80 mm or greater, longitudinal bars shall not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces, shall be provided, Lap splices shall conform to Article 12.17.

7.8.2 STEEL CORES

Load transfer in structural steel cores of composite compression members shall be provided by the following:

7.8.2.1 Ends of structural steel cores shall be accurately finished to bear at end bearing splices, with positive provision for alignment of one core above the other in concentric contact.

7.8.2.2 At end bearing splices, bearing shall be considered effective to transfer not more than 50 percent of the total compressive stress in the steel core.

7.8.2.3 Transfer of stress between column base and footing shall be designed in accordance with Article 15.8.

7.8.2.4 Base of structural steel section shall be designed to transfer the total load from the entire composite member to the footing; or, the base may be designed to transfer the load from the steel core only, provided ample concrete section is available for transfer of the portion of the total load carried by the reinforced concrete section to the footing by compression in the concrete and by reinforcement.

7.9 Connections

7.9.1 At connections of principal framing elements (such as beams and columns) enclosure shall be provided for splices of continuing reinforcement and for anchorage of reinforcement terminating in such connections.

7.9.2 Enclosure at connections may consist of external concrete or internal closed ties, spirals, or stirrups.

7.10 Lateral reinforcement for compression chambers.

7.10.1 Lateral reinforcement for compression members shall conform to the provisions of Articles 7.10.4 and 7.10.5 and, here shear or torsion reinforcement in required, shall also conform to provisions of Sub-section 11.

7.10.2 Lateral reinforcement requirements for composite compression members shall conform to Article 10.14. Lateral reinforcement requirements for pre-stressing tendons shall conform to Article 18.11.

7.10,3 Lateral reinforcement requirements of Articles 7.10, 10.14, and 18.11 may be waived where tests and structural analysis show adequate strength and feasability of construction.

7.10.4 SPIRALS

Spiral reinforcement for compression members shall conform to Article 10.9.3 and to the following:

7.10.4.1 Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled to permit handling and placing without distortion from designed dimensions.

7.10.4.2 For cast-in-place construction, size of spirals shall not be less than 10 mm diameter.

7.10.4.3 Clear spacing between spirals shall not exceed 80 mm nor be less than 25 mm. See also Article 3.3.3.

7.10.4.4 Anchorage of spiral reinforcement shall be provided by 1-1/2 extra turns of spiral bar or wire at each end of a spiral unit

7.10.4.5 Splices in spiral reinforcement shall be lap splices of $48d_b$ but not less than 300 mm, or welded.

7.10.4.6 Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

7.10.4.7 Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab or drop panel.

7.10.4.8 In columns with capitals, spirals shall extend to a level at which the diameter or width of capital is two times that of the column.

7.10.4.9 Spirals shall be held firmly in place and true to line by vertical spacers.

7.10.4.10 For spiral bar or wire smaller than 16 mm diameter, a minimum of two spacers shall be used for spirals less than 500 mm in diameter, three spacers for spirals 600 mm or less in diameter, and four spacers for spirals greater than 800 mm in diameter.

7.10.4.11 For spiral bar or wire 16 mm diameter or larger, a minimum of three spacers shall be used for spirals 600 mm or less in diameter, and four spacers for spirals greater than 600 mm in diameter.

7.10.5 TIES

Tie reinforcement for compression members shall conform to the following:

7.10.5.1 All non-pre-stressed bars shall be enclosed by lateral ties, at least 10 in size. Deformed wire or welded wire fabric of equivalent area may be used.

7.10.5.2 Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

7.10.5.3 Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 deg and no bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie may be used.

7.10.5.4 Ties shall be located vertically not more than 1/2 a tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than 1/2 a tie spacing below the lowest horizontal reinforcement in slab or drop panel above.

7.10.5.5 Where beams or brackets frame from four directions into a column, ties may be terminated not more than 80 mm below lowest reinforcement in shallowest of such beams or brackets.

7.11 Lateral reinforcement for flexural members

7.11.1 Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in Article 7.10.5 or by welded wire fabric of equivalent area. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.

7.11.2 Lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.

7.11.3 Closed ties or stirrups may be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced with a Class C splice (lap of 1.7 l_d), or anchord in accordance with Article 12.13.

7.12 Shrinkage and temperature reinforcement

7.12.1 Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement shall be provided in structural slabs where the flexural reinforcement extends in one direction only.

7.12.1.1 Shrinkage and temperature reinforcement shall be provided in accordance with Article 7.12.1 or 7.12.3.

7.12.2 Deformed reinforcement conforming to Article 3.5.3 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

7.12.2.1 Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0014:

(a) Slabs where Grade 300 deformed bars are used	0.0020
(b) Slabs where Grade 400 deformed bars or welded wire fabric (smooth or deformed) are used	0.0018

(c) Slabs where reinforcement with yield stress exceeding 400 MPa measured at a yield strain of 0.35 percent is used $\qquad \dfrac{0.0018 \times 400}{f_y}$

7.12.2.2 Shrinkage and temperature reinforcements shall be spaced not farther apart than five times the slab thickness, nor 500 mm.

7.12.2.3 At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the specified yield strength f_y in tension in accordance with Sub-section 12.

7.12.3 Pre-stressing tendons conforming to Article 3.5.5 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

7.12.3.1 Tendons shall be proportioned to provide a minimum average compressive stress of 1.0 MPa on gross concrete area using effective prestress, after losses, in accordance with Article 18.6.

7.12.3.2 Spacing of tendons shall not exceed 2 m.

7.12.3.3 When spacing of tendons exceeds 1.4 m, additional bonded shrinkage and temperature reinforcement conforming to Article 7.12.2 shall be provided between the tendons at slab edges extending from the slab edge for a distance equal to the tendon spacing.

PART 2

STRUCTURAL DESIGN REQUIREMENTS

SECTION 6D

REINFORCED AND PRE-STRESSED CONCRETE

GENERAL REQUIREMENTS

PART 2
SECTION 6 D

SUB-SECTION 8 - ANALYSIS AND DESIGN - GENERAL CONSIDERATIONS

8.1 Design methods

8.1.1 In design of reinforced concrete structures, members shall be proportioned for adequate strength in accordance with provision of this Code, using load factors and strength reduction factors ϕ specified in Sub-section 9.

8.1.2 Alternatively, non-pre-stressed reinforced concrete members may be designed using service loads stresses in accordance with provisions of Appendix B - Alternate Design Method.

8.2 Loading*

8.2.1 Design provisions of this Code are based on the assumption that structures shall be designed to resist all applicable loads.

8.2.2 Service loads shall be in accordance with the General Building Code of which this Code forms a part, with such live load reductions as are permitted in the General Building Code.

8.2.3 In design for wind and earthquake loads, integral structural parts shall be designed to resist the total lateral loads.**

8.2.4 Consideration shall be given to effects of forces due to pre-stressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, and unequal settlement of supports.

*Provisions in this Code are suitable for live, wind, and earthquake loads, such as those recommended in "Minimum Design Loads in Building and Other Structures." ANSI A-58.1 of the American National Standards Institute.

**Special provisions for seismic design are given in Appendix A.

8.3 Methods of analysis

8.3.1 All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified according to Article 8.4. Simplifying assumptions of Article 8.6 through 8.9 may be used.

8.3.2 Except for pre-stressed concrete, approximate methods of frame analysis may be used for buildings of usual types of construction, spans and story heights.

8.3.3 In lieu of a more accurate method of frame analysis, the following approximate moments and shears may be used in design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), provided:

(a) There are two or more spans,
(b) Spans are approximtely equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent.
(c) Loads are uniformly distributed.
(d) Unit live load does not exceed three times unit dead load, and
(e) Members are prismatic.

Positive Moment	
End Spans	
Discontinuous end unrestrained	$W_u l_n^2/11$
Discontinued end integral with support	$W_u l_n^2/14$
Interior spans	$W_u l_n^2/16$
Negative moment at exterior face of first interior support	
Two spans	$W_u l_n^2/9$
More than two spans	$W_u l_n^2/10$
Negative moment at other faces of interior supports	$W_u l_n^2/11$
Negative moment at face of all supports for:	
Slabs with spans not exceeding 3 m; and Beams where ratio of sum of column stiffnesses to beam stiffness exceeds eight at each end of the span	$W_u l_n^2/12$

Negative moment at interior face of exterior support for members build integrally with supports	
Where support is a spandrel beam	$W_u l_n^2/24$
Where support is a column	$W_u l_n^2/16$
Shear in end members at face of first interior support	$1.15\ W_u l_n/2$
Shear at face of all other supports	$W_u l_n/2$

8.4 Redistribution of negative moments in continuous non-pre-stressed flexural members*

8.4.1 Except where approximate values for moments are used, negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement may each be increased or decreased by not more than

$$20\left(1 - \frac{\rho - \rho'}{\rho_b}\right) \text{ percent}$$

8.4.2 The modified negative moments shall be used for calculating moments at sections within the spans.

8.4.3 Redistribution of negative moments shall be made only when the section at which moment is reduced is so designed that ρ or $\rho - \rho'$ is not greater than $0.50\rho_b$, where

$$\rho_b = \frac{0.85\beta_1 f'_c}{f_y}\frac{600}{600 + f_y} \qquad (8\text{-}1)$$

8.5 Modulus of elasticity

8.5.1 Modulus of elasticity E_c for concrete may be taken as $W_c^{1.5}\ 0.043\sqrt{f'_c}$ (in MPa) for values of W_c between 1500 and 2500 kg/m^3. For normal density concrete, E_c may be taken as $4700\sqrt{f'_c}$.

8.5.2 Modulus of elasticity E_s for non-pre-stressed reinforcement may be taken as 200,000 MPa.

8.5.3 Modulus of elasticity E_s for pre-stressing tendons shall be determined by tests or supplied by the manufacturer.

* For criteria on moment redistribution for pre-stressed concrete members, see Article 18.10.4

8.6 Stiffness

8.6.1 Any reasonable assumptions may be adopted for computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems. Assumptions shall be consistent throughout analysis.

8.6.2 Effect of haunches shall be considered both in determining moments and in design of members.

8.7 Span length

8.7.1 Span length of members not built integrally with supports shall be considered the clear span plus depth of member but need not exceed distance between centers of supports.

8.7.2 In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance center-to-center of supports.

8.7.3 For beams built integrally with supports, moments at faces of support may be used for design.

8.7.4 Solid or ribbed slabs built integrally with supports, with clear spans not more than 3 m, may be analysed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected.

8.8 Columns

8.8.1 Columns shall be designed to resist the axial forces from factored loads on all floors or roof and the maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Loading condition giving the maximum ratio of moment to axial load shall also be considered.

8.8.2 In frames or continuous construction, consideration shall be given to the effect of unbalanced floor or roof loads on both exterior and interior columns and of eccentric loading due to other causes.

8.8.3 In computing moments in columns due to gravity loading, far ends of columns built integrally with the structure may be considered fixed.

8.8.4 Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediatey above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

8.9 Arrangement of live load

8.9.1 Live load may be considered to be applied only to the floor or roof under consideration, and far ends of columns built integrally with the structure may be considered fixed.

8.9.2 Arrangement of live load may be limited to combinations of:

(a) Factored dead load on all spans with full factored live load on two adjacent spans, and

(b) Factored dead load on all spans with full factored live load on alternate spans.

8.10 T-beam construction

8.10.1 In T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

8.10.2 Width of slab effective as a T-beam flange shall not exceed one-quarter the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

(a) eight times the slab thickness, nor

(b) one-half the clear distance to the next web.

8.10.3 For beams with a slab on one side only, the effective overhanging flange width shall not exceed:

(a) one-twelfth the span length of the beam,

(b) six times the slab thickness, nor

(c) one-half the clear distance to the next web.

8.10.4 Isolated beams, in which the T-shape is used to provide a flange for additional compresssion area, shall have a flange thickness not less than one-half the width of the web and an effective flange width not more than four times the width of the web.

8.10.5 Where primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement perpendicular to the beam shall be provided in the top of the slab in accordance with the following:

8,10.5.1 Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width, assumes to act as a cantiliver. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective overhanging slab width need be considered.

8.10.5.2 Transverse reinforcement shall be spaced not farther apart than five times the slab thickness, nor 500 mm.

8.11 Joist construction

8.11.1 Joist construction consists of a monolithic combination of reguarly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

8.11.2 Ribs shall be not less than 100 mm in width; and shall have a depth of not more than 3-1/2 times the minimum width of rib.

8.11.3 Clear spacing between the ribs shall not exceed 800 mm.

8.11.4 Joist construction not meeting the limitations of Articles 8.11.1 through 8.11.3 shall be designed as slabs and beams.

8.11.5 When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to that of the specified strength of concrete in the joists are used:

8.11.5.1 Vertical shells of fillers in contact with the ribs may be included in strength computations for shear and negative moment. Other portions of fillers shall not be included in strength computations.

8.11.5.2 Slab thickness over permanent fillers shall not be less than one-twelfth the clear distance between ribs, nor less than 40 mm.

8.11.5.3 In one-way joists, reinforcement normal to the ribs shall be provided in the slab as required by Article 7.12.

8.11.6 When removable forms or fillers not complying with Article 8.11.5 are used:

8.11.6.1 Slab thickness shall not be less than one-twelfth the clear distance between ribs, nor less than 50 mm.

8.11.6.2 Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations, if any, but not less than required by Article 7.12.

8.11.7 Where conduits or pipes as permitted by Article 6.3 are embedded within the slab, slab thickness shall be at least 25 mm greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not impair significantly the strength of the construction.

8.11.8 Shear strength provided by concrete V_c for the ribs may be taken as 10 percent greater than provided in Chapter 11. Shear strength may be increased by use of shear reinforcement or by widening the ends of the ribs.

8.12 Separate floor finish

8.12.1 A floor finish shall not be included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with requirements of Chapter 17.

8.12.2 All concrete floor finishes may be considered as part of the required cover or total thickness for non-structural considerations.

SUB-SECTION 9 - STRENGTH AND SERVICEABILITY REQUIREMENTS

9.1 General

9.1.1 Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this Code.

9.1.2 Members also shall meet all other requirements of this Code to insure adequate performance at service load levels.

9.2 Required strength

9.2.1 Required strength U to resist dead load D and live load L shall be at least equal to

$$U = 1.4D + 1.7L \tag{9-1}$$

9.2.2 If resistance to structural effects of a specified wind load W are included in design, the following combinations of D, L, and W shall be investigated to determine the greatest required strength U

$$U = 0.75(1.4D + 1.7L + 1.7W) \tag{9-2}$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.3W \tag{9-3}$$

but for any combination of D,L, and W, required strength U shall not be less that Eq. (9-1).

9.2.3 If resistance to specified earthquake loads or forces E are included in design, load combinations of Article 9.2.2 shall apply, except that 1.1E shall be substituted for W.

9.2.4 If resistance to earth pressure H is included in design, required strength U shall be at least equal to

$$U = 1.4D + 1.7L + 1.7H \tag{9-4}$$

except that where D or L reduce the effect of H, 0.9D shall be substituted for 1.4D and zero value of L shall be used to determine the greatest required strength U. For any combination of D, L and H, required strength U shall not be less than Eq. (9-1).

9.2.5 If resistance to loadings due to weight and pressure of fluids with well-defined densities and controlled maximum heights F is included in design, such loading shall have a load factor of 1.4 and be added to all loading combinations that include live load.

9.2.6 If resistance to impact effects is taken into account in design, such effects shall be included with live load L.

9.2.7 Where structural effects of T or differential settlement, creep, shrinkage, or temperature change may be significant in design, required strength U shall be at least equal to

$$U = 0.75(1.4D + 1.4T + 1.7L) \tag{9-5}$$

but required strength U shall not be less than

$$U = 1.4(D + T) \tag{9-6}$$

Estimations of differential settlement, creep, shrinkage, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.3 Design strength

9.3.1 Design strength provided by a member, its connections to other members, and its cross-sections, in terms of flexure, axial load, shear and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this Code, multiplied by a strength reduction factor ϕ.

9.3.2 Strength reduction factor ϕ shall be as follows:

9.3.2.1 Flexure, without axial load 0.90

9.3.2.2 Axial load, and axial load with flexure. (For axial load with flexure, both axial load and moment, nominal strength shall be multiplied by appropriate single value of ϕ).

(a) Axial tension, and axial tension with flexure 0.90

(b) Axial compression, and axial compression with flexure:
Members with spiral reinforcement conforming to Article 10.9.3 ......0.75
Other reinforced members...........0.70

except that for low values of axial compression may be increased in accordance with the following:

For members in which f_y, does not exceed 400 MPa with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10f'_c A_g$ to zero.

For other reinforced members, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10f'_c A_g$ or $0P_b$ whichever is smaller, to zero.

9.3.2.3 Shear and torsion 0.85

9.3.2.4 Bearing on concrete (See also Article 18.13) 0.70

9.3.3 Development lengths specified in Chapter 12 do not require a 0 factor.

9.4 Design strength for reinforcement

Designs shall not be based on a yield strength of reinforcement f_y in excess of 550 MPa, except for pre-stressing tendons.

9.5 Control of deflections

9.5.1 Reinforced concrete members subject to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect strength or serviceability of a structure at service loads.

9.5.2 One-way construction (non-pre-stressed)

9.5.2.1 Minimum thickness stipulated in Table 9.5(a) shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness may be used without adverse effects.

9.5.2.2 Where deflections are to be computed, deflections that occur immediately on application of load shall be computed by usual methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

TABLE 9.5(A)
MINIMUM THICKNESS OF NON-PRE-STRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED*

	Minimum thickness, h			
	Simply Supported	One-end con-tinuous	Both-ends con-tinuous	Canti-lever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	1/20	1/24	1/28	1/10
Beams or ribbed one-way slabs	1/16	1/18.5	1/21	1/8

*Span length l is in millimeters.

Values given shall be used directly for members with normal density concrete (w_c = 2300 kg/m^3) and Grade 400 reinforcement. For other conditions, the values shall be modified as follows:
(a) For structural low-density concrete having masses in the range 1500 - 2000 kg/m^3, the values shall be multiplied by (1.65 - 0.005 w_c) but not less than 1.09, where w_c is the unit mass in kg/m^3.
(b) For f_y other than 400 MPa the values shall be multiplied by $(0.4 + f_y/700)$.

9.5.2.3 Unless stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be computed with the modulus of elasticity E_c for concrete as specified in Article 8.5.1. (normal density or low density concrete) and with the effective moment of inertia as follows, but not greater than I_g.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \quad (9\text{-}7)$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (9\text{-}8)$$

and for normal density concrete

$$f_r = 0.7\sqrt{f'_c} \qquad (9\text{-}9)$$

When low-density aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified and concrete is proportioned in accordance with Article 4.2, f_r shall be modified by substituting $1.8f_{ct}$ for $\sqrt{f'_c}$ but the value of $1.8f_{ct}$ shall not exceed $\sqrt{f'_c}$.

(b) When f_{ct} is not specified, f_r shall be multiplied by 0.75 for "all-low-density" concrete, and 0.85 for "sand-low density" concrete. Linear interpolation may be used when partial sand replacement is used.

9.5.2.4 For continuous members, effectve moment of inertia may be taken as the average of values obtained from Eq. (9-7) for the critical positive and negative moment sections. For prismatic members, effective moment of inertia may be taken as the value obtained from Eq. (9-7) at mid-span for simple and continuous spans, and at support for cantilevers.

9.5.2.5 Unless values are obtained by a more comprehensive analysis, additional long-time deflection resulting from creep and shrinkage of flexural members (normal density or low-density concrete) shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor

$$\lambda = \frac{\xi}{1+50\rho'} \qquad (9\text{-}10)$$

where ρ' shall be the value at midspan for simple and continuous span, and at support for cantilevers. Time-dependent factor ξ for sustained loads may be taken equal to

5 years or more	2.0
12 months	1.4
6 months	1.2
3 months	1.0

9.5.2.6 Deflection computed in accordance with Articles 9.5.2.2 through 9.5.2.5 shall not exceed limits stipulated in Table 9.5(b).

9.5.3 TWO-WAY CONSTRUCTION (NON-PRE-STRESSED)

9.5.3.1 Minimum thickness of slabs or other two-way construction designed in accordance with provisions of Chapter 13, and having a ratio of long to short span not exceeding 2, shall be governed by Eq. (9-11), (9-12) and (9-13) and the other provisions of Article 9.5.3.

$$h = \frac{\ell_n(800 + f_y/1.5)}{36{,}000 + 5000\beta\left[\alpha_m - 0.5(1 - \beta_s)\left(1 + \frac{1}{\beta}\right)\right]} \qquad (9\text{-}11)$$

but not less than

$$h = \frac{\ell_n(800 + f_y/1.5)}{36{,}000 + 5000\beta(1 + \beta_s)} \qquad (9\text{-}12)$$

and need not be more than

$$h = \frac{\ell_n(800 + f_y/1.5)}{36{,}000} \qquad (9\text{-}13)$$

however, the thickness shall not be less than the following values:

(a) Slabs without beams or drop panels. . . . . 120 mm

(b) Slabs without beams, but with drop panels conforming to Article 9.5.3.2. . . . . . 100 mm

(c) Slabs with beams, on all four edges with a value of α_m at least equal to 2.0. . . . . 90 mm

TABLE 9.5(B)
MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\frac{1}{180}$ *
Floors not supported or attached to nonstructural elements likely to be damaged by large deflections.	Immediate deflection due to live load L.	$\frac{1}{360}$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-time deflection due to all sustained loads and the immediate deflection due to any additional live load.) ***	$\frac{1}{480}$ **
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections.		$\frac{1}{240}$ ****

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-time effects of all sustained loads, construction tolerances and reliability of provisions for drainage.

** Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

*** Long-time deflection shall be determined in accordance with Article 9.5.2.5 or 9.5.4.2 but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

**** But not greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

9.5.3.2 For slabs without beams, but with drop panels extending in each direction from centreline of support a distance of not less than one-sixth the span length in that direction measured centre-to-centre of supports, and a projection below the slab at least one-quarter the slab thickness beyond the drop, thickness required by Eq.(9-11), (9-12) or (9-13) may be reduced by 10 percent.

9.5.3.3 At discontinuous edges, an edge beam shall be provided with stiffness ratio not less than 0.80; or the minimum thickness required by Eq.(9-11), (9-12), (9-13) or Article 9.5.3.2 shall be increased by at least 10 percent in the panel with a discontinuous edge.

9.5.3.4 Slab thickness less than the minimum thickness required by Articles 9.5.3.1, 9.5.3.2, and 9.5.3.3 may be used if shown by computation that deflection will not exceed the limits stipulated in Table 9.5(b). Deflections shall be computed taking into account size and shape of panel, conditions of support, and nature of restraints at panel edges. For deflection computations, modulus of elasticity E_c for concrete shall be as specified in Article 8.5.1. Effective moment of inertia shall be that given by Eq. (9-7); other values may be used if computed deflection is in reasonabe agreement with results of comprehensive tests. Additional long-time deflection shall be computed in accordance with Article 9.5.2.5.

9.5.4 PRE-STRESSED CONCRETE CONSTRUCTION

9.5.4.1 For flexural members designed in accordance with provision of Chapter 18, immediate deflection shall be computed by usual methods or formulas for elastic deflections, and the moment of inertia of the gross concrete section may be used for uncracked sections.

9.5.4.2 Additional long-time deflection of pre-stressed concrete members shall be computed taking into account stresses in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of steel.

9.5.4.3 Deflection computed in accordance with Articles 9.5.4.1 and 9.5.4.2 shall not exceed limits stipulated in Table 9.5(b).

9.5.5 COMPOSITE CONSTRUCTION

9.5.5.1 Shored construction

If composite flexural members are supported during construction so that, after removal of temporary supports, deal load is resisted by the full composite section, the composite member may be considered equivalent to a monolithically cast member for computation of deflection. For non-pre-stressed members, the portion of the member in compression shall determine whether values in Table 9.5(a) for normal density or low-density concrete shall apply. If deflection is computed, account should be taken of curvatures resulting from differential shrinkage of precast and cast-in-place components, and of axial creep effects in a pre-stressed concrete member.

9.5.5.2 If the thickness of a non-pre-stressed precast flexural member meets the requirements of Table 9.5(a), deflection occurring after the member becomes composite need not be computed, but the long-time deflection of the precast member should be investigated for magnitude and duration of load prior to beginning of effective composite action.

9.5.5.3 Deflection computed in accordance with Articles 9.5.5.1 and 9.5.5.2 shall not exceed limits stipulated in Table 9.5(b).

9.5.6 LONG-TERM DEFLECTIONS

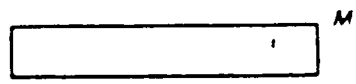
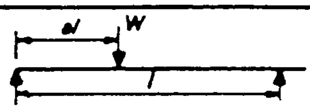
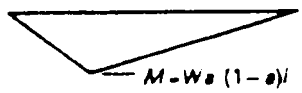
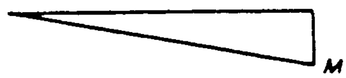
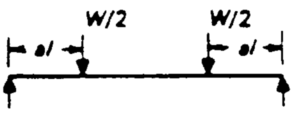
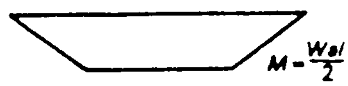
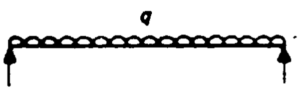
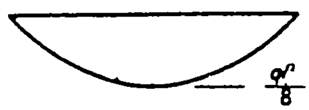
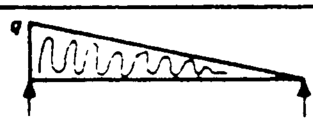
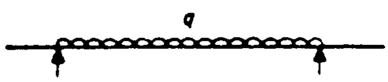
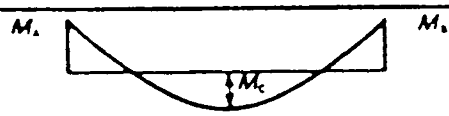
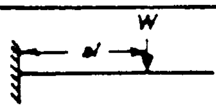
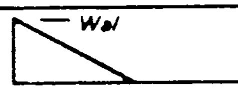
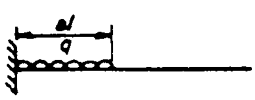
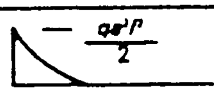
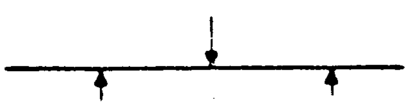
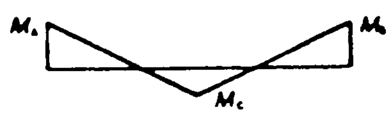
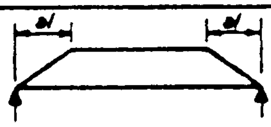
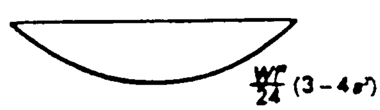
9.5.6.1 Deflections are calculated from curvature of sections under appropriate moments The deflection a, at any point on a member is calculated by the equation
$a = Kl^2 \times 1/r_b$ where

l = effective span of the member

r_b^{-1} = curvature of the member at mid-span or for cantilevers at the support section

K = constant which depends on the shape of the bending moment diagram.

Values for K for various B.M. diagrams are tabulated below:

Loading	Bending Moment Diagram	K
		0.125
		$\frac{4a^2 - 8a - 1}{48a}$ if $a = \frac{1}{2}$ $K = \frac{1}{12}$
		0.0625
		$0.125 - \frac{a^2}{6}$
		0.104
		0.102
		$K = 0.104\left(1 - \frac{\beta}{10}\right)$ $\beta = \frac{M_A + M_B}{M_C}$
		end deflection $= \frac{a(3-a)}{6}$ load at end $K = 0.333$
		$\frac{a(4-a)}{12}$ if $a = l$ $K = 0.25$
		$K = 0.083\left(1 - \frac{\beta}{4}\right)$ $\beta = \frac{M_A + M_B}{M_C}$
		$\frac{1}{80}\ \frac{(5-4a^2)^2}{3-4a^2}$

9.5.6.2 Calculation of curvatures

The total long term curvature of a section can be assessed as follows:

(a) Calculate the instantaneous curvature under the total load and under permanent load.
(b) Calculate the long term curvature under permanent load.
(c) Add to the long term curvature under permanent load the difference between the instantaneous curvature under the total and permanent load.
(d) Add to this curvature the shrinkage curvature.

9.5.6.2.1 Calculation of curvatures

The curvature of any section may be calculated by employing whichever of the following sets of assumptions (A) or (B) gives the larger value. (A) corresponds to the case where the section is cracked under the loading considered, (B) applies to an uncracked section.

(A) (1) Strains are calculated on the assumption that plane sections remain plane.

(2) The reinforcement, whether in tension or in compression, is assumed to be elastic. Its modulus of elasticity may be taken as 200 kN/mm^2.

(3) The concrete in compression is assumed to be elastic. Under short term loading, the modulus of elasticity may be taken as that given in 8.5.1. Under long term loading, an effective modulus may be taken having a value of $1/(1+\varphi)$ times the short term modulus where φ is the appropriate creep coefficient.

(4) Stresses in the concrete in tension may be calculated on the assumption the stress distribution is triangular, having a value of zero at the neutral axis and a value at the centroid of the tension steel of 1 N/mm^2 instantaneously, reducing to 0.55 N/mm^2 in the long term.

(B) The concrete and the steel are both considered to be fully elastic in tension and in compression. The elastic modulus of the steel may be taken as 200 kN/mm^2 and the elastic modulus of the concrete is as specified in (3) above both in compression and in tension.

Creep coefficient φ and stress in concrete at tensile steel level:

Duration of load days	Age at loading 7	14	28	56	100	Pseudo-cracked tensile stress f_cK (N/mm^2)
0	0	0	0	0	0	1.00
10	.39	.33	.27	.23	.19	.95
100	1.92	1.65	1.37	1.37	1.17	.96
1000	3.08	2.64	2.20	1.87	1.54	.64
	3.85	3.30	2.75	2.34	1.93	.55

9.5.6.2.2 Shrinkage curvature

The shrinkage curvature is calculated from the formula

the shrinkage curvature and ϵ_{cs} in the free shrinkage strain in the assessment of which allowance is made for the shape of the section as follows:

Sections less than 250 mm thick $\epsilon_{cs} = 300 \times 10^{-6}$

Sections thicker than 250 mm $\epsilon_{cs} = 250 \times 10^{-6}$

Values of ρ_o for calculation of shrinkage curvatures

values of ρ	values of ρ' 0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00
0.25	0.44	0.31	0.26	0.22	0.20	0.18	0.17	0.16	0.15
0.50	0.56	0.31	0.26	0.22	0.20	0.18	0.17	0.16	0.15
0.75	0.64	0.45	0.26	0.22	0.20	0.18	0.17	0.16	0.15
1.00	0.70	0.55	0.39	0.22	0.20	0.18	0.17	0.16	0.15
1.50	0.80	0.69	0.57	0.45	0.32	0.18	0.17	0.16	0.15
2.00	0.88	0.79	0.69	0.60	0.49	0.39	0.28	0.16	0.15
2.50	0.95	0.87	0.79	0.70	0.62	0.53	0.44	0.35	0.25
3.00	1.00	0.94	0.86	0.79	0.72	0.64	0.57	0.49	0.40
3.50	1.00	1.00	0.93	0.87	0.80	0.74	0.67	0.60	0.52
4.00	1.00	1.00	1.00	0.93	0.87	0.81	0.75	0.69	0.62

Percentage shrinkage occurring up to a given age:

Age days	10	28	100	365	1000	∞
%	10	30	50	70	80	100

9.6 Crackwidths

9.6.1 The widths of flexural cracks at a particular point on the surface of a member depends primarily on three (3) factors:

(a) the proximity to the point considered of the reinforcing bar perpendicular to the cracks;
(b) the proximity of the neutral axis to the point considered;
(c) the average strain at the point considered.

9.6.2 Crackwidth evaluation

9.6.2.1 One simplification based on a structural study of test datas of several investigators is the

$$W_{max} = 0.0110\ \beta f_s \sqrt[3]{d_c A}$$

where Wmax = crackwidth in units of 0.001 mm

$\beta = (h-c)/(d_c - c)$ = depth factor average value = 1.20

d_c = thickness of cover to the centre of the first layer of bars (mm)

f_s = maximum stress in N/mm^2 in the steel at service beds with 0.6 f_y to be used if no computations are available

A = area of concrete in tension divided by the number of bars (mm^2) = b_t/b_c where b_c is defined as the number of bars at the tension side.

9.6.3 PERMISSIBLE CRACKWIDTHS

9.6.3.1 The maximum crackwidth that a structural element should be permitted to develop depends on the particular function of the element and the environmental conditions to which the structure is liable to be subjected. ACI Committee 224 recommends the following:

Permissible Crackwidths

Exposure Conditions	Tolerable crackwidths
Dry air or protective membrane	0.41 mm
Humidity, moist air, soil	0.30 mm

Seawater, and seawater spray Wetting and drying	0.15 mm
Water-retaining structures (excluding non-pressure pipes)	0.10 mm

SUB-SECTION 10 - FLEXURE AND AXIAL LOADS

10.1 Scope

Provisions of Chapter 10 shall apply for design of members subject to flexure or axial loads or to combined flexure and axial loads.

10.2 Design assumptions

10.2.1 Strength design of members for flexure and axial loads shall be based on assumptions given in Articles 10.2.2 through 10.2.7 and on satisfaction of applicable conditions of equilibrium and compatibility of strains.

10.2.2 Strain in reinforcement and concrete shall be assumed directly proportional to the distance from the neutral axis, except for deep flexural members with overall depth to clear span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a non-linear distribution of strain shall be considered. See Article 10.7.

10.2.3 Maximum usable strain at extreme concrete compression fibre shall be assumed equal to 0.003.

10.2.4 Stress in reinforced below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that correspondig to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

10.2.5 Tensile strength of concrete shall be neglected in flexural calculations of reinforced concrete, except when meeting requirements of Article 18.4.

10.2.6 Relationship between concrete compressive stress distribution and concrete strain may be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

10.2.7 Requirements of Article 10.2.6 may be considered satisfied by an equivalent rectangular concrete stress distribution defined by the following:

10.2.7.1 Concrete stress of $0.85f'_c$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain.

10.2.7.2 Distance c from fibre of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

10.2.7.3 Factor β_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 30 MPa. For strengths above 30 MPa, β_1 shall be reduced continuously at a rate of 0.008 for each 1 MPa of strength in excess of 30 MPa, but β_1 shall not be taken less than 0.65.

10.3 General priniciples and requirements

10.3.1 Design of cross section subject to flexure or axial loads or to combined flexure and axial loads shall be based on stress and strain compatibility using assumptions in Article 10.2.

10.3.2 Balanced strain conditions exist at a cross-section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.

10.3.3 For flexural members, and for members subject to combined flexure and compressive axial load when the design axial load strength ϕP_n is less than the smaller of $0.10f'_c A_g$ or ϕP_b, the ratio of reinforcement ρ provided shall not exceed 0.75 of the ratio ρ_b that would produce balanced strain conditions for the section under flexure without axial load. For members with compression reinforcement, the portion of ρ_b equalized by compression reinforcement need not be reduced by the 0.75 factor.

10.3.4 Compression reinforcement in conjunction with additional tension reinforcement may be used to increase the strength of flexural members.

10.3.5 Design axial load strength ϕPn of compression members shall not be taken greater than the following:

10.3.5.1 For non-pre-stressed members with spiral reinforcement conforming to Article 7.10.4 or composite members conforming to Article 10.14.

$$\phi P_{n(max)} = 0.85\phi[0.85f'_c(A_g - A_{st}) + f_y A_{st}] \quad (10\text{-}1)$$

10.3.5.2 For non-pre-stressed members with tie reinforcement conforming to Article 7.10.5:

$$\phi P_{n(max)} = 0.80\phi[0.85f'_c(A_g - A_{st}) + f_y A_{st}] \quad (10\text{-}2)$$

10.3.5.3 For pre-stressed members, design axial load strength ϕP_n shall not be taken greater than 0.85 (for members with spiral reinforcement) or 0.80 (for members with tie reinforcement) of the design axial load strength at zero eccentricity ϕP_o.

10.3.6 Members subject to compressive axial load shal be designed for the maximum moment that can accompany the axial load. The factored axial load P_u at given eccentricity shall not exceed that given in Article 10.3.5. The maximun factored moment M_u shall be magnified for slenderness effects in accordance with Article 10.10.

10.4 Distance between lateral supports of flexural members

10.4.1 Spacing of lateral supports for a beam shall not exceed 50 times the least width b of compression flange or face.

10.4.2 Effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

10.5 Minimum reinforcement of flexural members

10.5.1 At any section of a flexural member, except as provided in Articles 10.5.2 and 10.5.3, where positive reinforcement is required by analysis, the ratio ρ provided shall not be less than that given by

$$\rho_{min} = \frac{1.4}{f_y} \qquad (10\text{-}3)$$

in T-beams and joists where the web is in tension, the ratio ρ shall be computed for this purpose using width of web.

10.5.2 Alternatively, the area of reinforcement provided at every section, positive or negative, shall be at least one-third greater that that required by analysis.

10.5.3 For structural slabs of uniform thickness, the minimum area and maximum spacing of reinforcement in the direction of the span shall be as required for shrinkage and temperature according to Article 7.12.

10.6 Distribution of flexural reinforcement in beams and one-way slabs

10.6.1 This section prescribes rules for distribution of flexural reinforcement to control flexural cracking in beams and in one-way slabs (slabs reinforce to resist flexural stresses in only one direction).

10.6.2 Distribution of flexural reinforcement shall be well distributed within maximum flexural tension zones of a member cross-section as required by Article 10.6.4.

10.6.3 Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross-section as required by Article 10.6.4.

10.6.4 When design yield strength f_y for tension reinforcement exceeds 300 MPa, cross sections of maximum positive and negative moment shall be so proportioned that the quantity given by

$$z = f_s \sqrt[3]{d_c A} \qquad (10\text{-}4)$$

does not exceed 30 MN/m for interior exposure and 20 MN/m for exterior exposure. Calculated stress in reinforcement at service load f_s (MPa) shall be computed as the moment divided by the product of steel area and internal moment arm. In lieu of such computations, f_s may be taken as 60 percent of specified yield strength f_y.

10.6.5 Provisions of Article 10.6.4 may not be sufficient for structures subject to very aggressive exposure or designed to be watertight. For such structures special investigations and precautions are required.

10.6.6 Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width as defined in Article 8.10, or a width equal to 1/10 the span, whichever is smaller, if the effective flange width exceeds 1/10 the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.

10.6.7 If the depth of a web exceeds 900 mm, longitudinal reinforcement having a total area equal to at least 10 percent of the area of the flexural tension reinforcement shall be placed near the side faces of the web and distributed in the zone of flexural tension with a spacing not more than the web width,nor 300 mm. Such reinforcement may be included in strength computations only if a strain compatibility analysis is made to determine stresses in the individual bars or wires.

10.7 Deep flexural members

10.7.1 Flexural members with overall depth to clear span ratios greater than 2/5 for continuous spans, or 4/5 for simple spans, shall be designed as deep flexural members taking into account non-linear distribution of strain and lateral buckling. (See also Article 12.10.6).

10.7.2 Shear strength of deep flexural members shall be in accordance with Article 11.8.

10.7.3 Minimum flexural tension reinforcement shall conform to Article 10.5.

10.7.4 Minimum horizontal and vertical reinforcement in the side faces of deep flexural members shall be the greater of the requirements of Articles 11.8.8 and 11.8.9 or Articles 14.3.2 and 14.3.3.

10.8 Design dimensions for compression members

10.8.1 Isolated compression member with compression members

Outer limits of the effective cross-section of a compression member with two or more interlocking spirals shall be taken at a distance outside the extreme limits of the spirals equal to the minimum concrete cover required by Article 7.7.

10.8.2 COMPRESSION MEMBER BUILT MONOLITHICALLY WITH WALL

Outer limits of the effective cross-section of a spirally reinforced or tied reinforced compression member built monolithically with a concrete wall or pier shall be taken not greater than 40 mm outside the spiral or tie reinforcement

10.8.3 EQUIVALENT CIRCULAR COMPRESSION MEMBER

In lieu of using full gross area for design, a compression member with a square octagonal, or other shaped cross-section may be considered as a circular section with a diameter equal to the least lateral dimension of the actual shape. Gross area considered, required percentage of reinforcement and design strength shall be based on that circular section.

10.8.4 LIMITS OF SECTION

For a compression member with a larger cross-section than required by considerations of loading, a reduced effective area A_g not less than one-half the total area may be used to determine minimum reinforcement and design strength.

10.9 Limits for reinforcement of compression members

10.9.1 Area of longitudinal reinforcement for non-composite compression members shall be not less than 0.01 nor more than 0.08 times gross area A_g of section.

10.9.2 Minimum number of longitudinal bars in compression members shall be 4 for bars within rectangular or circular ties, 3 for bars within triangular ties, and 6 for bars enclosed by spirals conforming to Article 10.9.3.

10.9.3 Ratio of spiral reinforcement ρ_s shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \qquad (10\text{-}5)$$

where f_y is the specified yield strength of spiral reinforcement but not more than 400 MPa.

10.10 Slenderness effects in compression members

10.10.1 Design of compression members shall be based on forces and moments determined from analysis of the structure. Such analysis shall take into account influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, effect of deflections on moments and forces, and the effects of duration of loads.

10.10.2 In lieu of the procedure described in Article 10.10.1, slenderness effects in compression members may be evaluated in accordance with the approximate procedure presented in Article 10.11.

10.10.3 The detailed requirements of Article 10.1 need not be applied if slenderness effects in compression members are evaluated in accordance with Article 10.10.1.

10.11 Approximate evaluation of slenderness effects

10.11.1 UNSUPPORTED LENGTH OF COMPRESSION MEMBERS

10.11.1.1 Unsupported length l_u of a compression member shall be taken as the clear distance between floor slabs, beams, or other members, capable of providing lateral support for that compression member.

10.11.1.2 Where column capitals or haunches are present, unsupported length shall be measured to the lower extremity of capital or haunch in the plane considered.

10.11.2 EFFECTIVE LENGTH OF COMPRESSION MEMBERS

10.11.2.1 For compression members braced against sidesway, effective length factor k shall be taken as 1.0 unless analysis shows that a lower value may be used.

10.11.2.2 For compression members not braced against sidesway, effective length factor k shall be determined with due consideration of effects of cracking and reinforcement on relative stiffness, and shall be greater than 1.0.

10.11.3 RADIUS OF GYRATION

Radius of gyration r may be taken equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members, and 0.25 times the diameter for circular compression members. For other shapes, r may be computed for the gross concrete section.

10.11.4 CONSIDERATION OF SLENDERNESS EFFECTS

10.11.4.1 For compression members braced against sidesway, effects of slenderness may be neglected when kl_u/r is less than $34-12M_{1b}/M_{2b}$.

10.11.4.2 For compression members not braced against sidesway, effects of slenderness may be neglected when kl_u/r is less than 22.

10.11.4.3 For all compression members with kl_u/r greater than 100, an analysis as defined in Article 10.10.1 shall be made.

10.11.5 MOMENT MAGNIFICATION

10.11.5.1 Compression members shall be designed using the factored axial load P_u from a conventional frame analysis and a magnified factored moment M_c defined by

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \qquad (10\text{-}6)$$

where

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} \geqslant 1.0 \qquad (10\text{-}7)$$

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_u}{\phi \Sigma P_c}} \geqslant 1.0 \qquad (10\text{-}8)$$

and $$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \qquad (10\text{-}9)$$

$\sum P_u$ and $\sum P_c$ are the summations for all columns in a story. For frames not braced against sidesway, both δ_b and δ_s shall be computed. For frames braced against sidesway, δ_s shall be taken as 1.0 in calculation of P_c, k shall be computed according to Article 10.11.2.1 for δ_b and according to Article 10.11.2.2 for δ_s.

10.11.5.2 In lieu of a more accurate calculation circulation, EI in Eq (10-9) may be taken either as

$$EI = \frac{(E_c I_g/5) + E_s I_{se}}{1 + \beta_d} \qquad (10\text{-}10)$$

or conservatively

$$EI = \frac{E_c I_g/2.5}{1 + \beta_d} \qquad (10\text{-}11)$$

10.11.5.3 In Eq. (10-7) for members braced against sidesway and without transverse loads between supports, C_m may be taken as

$$C_m = 0.6 + 0.4\frac{M_{1b}}{M_{2b}} \qquad (10\text{-}12)$$

but not less than 0.4.

For all other cases, C_m may be taken as 1.0.

10.11.5.4 If computations show that there is no moment at both ends of a braced compression member or that computed end eccentricities are less than (15 + 0.03h) mm, M_{2b} in Eq. (10-6) shall be based on a minimum eccentricity of (15 + 0.03h) mm about each principal axis separately. Ratio M_{1b}/M_{2b} in Eq. (10-12) shall be determined by either of the following:

(a) When computed end eccentricities are less than (15 + 0.03h)mm, computed end moments may be used to evaluate M_{1b}/M_{2b} in Eq. (10-12)

(b) If computations show that there is essentially no moment at both ends of a compression member, the ratio M_{1b}/M_{2b} shall be taken equal to one.

10.11.5.5 If computations show that there is no moment at both ends of a compression member not braced against sidesway or that computed end eccentricities are less than (15 + 0.03h)mm, M_{2s} in Eq. (10-6) shall be based on a minimum eccentricity of (15 + 0.03h)mm about each principal axis separately.

10.11.6 MOMENT MAGNIFICATION FOR FLEXURAL MEMBERS

In frames not braced against sidesway, flexural members shall be designated for the total magnified end moments of the compression members of the joint.

10.11.7 MOMENT MAGNIFIER δ FOR BI-AXIAL BENDING

For compression members subject to bending about both principal axis, moment about each axis shall be magnified by δ, computed from corresponding conditions of restraint about that axis.

10.12 Axially loaded members supporting slab system

Axially loaded members supporting a slab system included within the scope of Article 13.1 shall be designed as provided in Chapter 10 and in accordance with the additional requirements of Chapter 13.

10.13 Transmission of column loads through floor system

When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be provided by one of the following:

10.13.1 Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 600 mm into the slab from face to column. Column concrete shall be well integrated with floor concrete, and shall be placed in accordance with Articles 6.4.5 and 6.4.6.

10.13.2 Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

10.13.3 For columns laterally supported on four sides by beams of approximately equal depth or by slabs, strength of the column may be based on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of foor concrete strength.

10.14 Composite compression members

10.14.1 Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe,or tubing with or without longitudinal bars.

10.14.2 Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

10.14.3 Any axial load strength assigned to concrete of composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.14.4 All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.14.5 For evaluation of slenderness effects, radius of gyration of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g/5) + E_s I_t}{(E_c A_g/5) + E_s A_t}} \qquad (10\text{-}13)$$

in lieu of a more accurate calculation EI in Eq (10-9) may be taken either as Eq. (10-11) or

$$EI = \frac{(E_c I_g/5)}{1 + \beta_d} + E_s I_t \qquad (10\text{-}14)$$

10.14.6 STRUCTURAL STEEL ENCASED CONCRETE CORE

10.14.6.1 For a composite member with concrete core enclosed by structural steel, thickness of the steel encasement shall be not less than

$$b\sqrt{\frac{f_y}{3E_s}}, \text{ for each face of width b}$$

nor

$$h\sqrt{\frac{f_y}{8E_s}}, \text{ for circular sections of diamter h}$$

10.14.6.2 Longitudinal bars located within the encased concrete core may be considered in computing A_t and I_t.

10.14.7 SPIRAL REINFORCEMENT AROUND STRUCTURAL STEEL CORE

A composite member with spirally reinforced concrete around a structural steel core shall conform to the following.

10.14.7.1 Specified compressive strength of concrete f'_c shall be not less than 17 MPa.

10.14.7.2 Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 350 MPa.

10.14.7.3 Spiral reinforcement shall conform to Article 10.9.3.

10.14.7.4 Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.14.7.5 Longitudinal bars located within the spiral may be considered in computing A_t and I_t.

10.14.8 TIE REINFORCEMENT AROUND STRUCTURAL STEEL CORE

A composite member with laterally tied concrete around a structural steel core shall conform to the following.

10.14.8.1 Specified compressive strength of concrete f'_c shall be not less than 17 MPa.

10.14.8.2 Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 350 MPa.

10.14.8.3 Lateral ties shall extend completely around the structural steel core.

10.14.8.4 Lateral ties shall have a diameter not less than 1/50 times the greatest side dimension of composite member, except that ties shall not be smaller than no. 10 and need not be larger than no. 15. Welded wire fabric of equivalent area may be used.

10.14.8.5 Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 1/2 times the least side dimension of the composite member.

10.14.8.6 Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.14.8.7 A longitudinal bar shall be located at every corner of a rectangular cross-section, with other longitudinal bars spaced not farther apart than one-half the least side dimension of the composite member.

10.14.8.8 Longitudinal bars located within the ties may be considered in computing A_t for strength but not in computing I_t for evaluation of slenderness effects.

10.15 **Bearing strength**

10.15.1 Design bearing strength on concrete shall not exceed $\phi(0.85f'_c A_1)$, except as follows.

10.15.1.1 When the supporting surface is wider on all sides than the loaded area, design bearing strength on the loaded area may be multiplied by $\sqrt{A_2/A_1}$, but not more than 2.

10.15.1.2 When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

10.15.2 Article 10.15 does not apply to post-tensioning anchorages.

SUB-SECTION 11 SHEAR AND TORSION

11.1 Shear strength

11.1.1 Design of cross-sections subject to shear shall be based on

$$V_u \leq \phi V_n \tag{11-1}$$

where V_u is factored shear force at section considered and V_n is nominal shear strength computed by

$$V_n = V_c + V_s \tag{11-2}$$

where V_u is nominal shear strength provided by concrete in accordance with Article 11.3 or 11.4 and V_s is nominal shear strength provided by shear reinforcement in accordance with Article 11.5.6.

11.1.1.1 In determining shear strength V_n, effect of any openings in members shall be considered.

11.1.1.2 In determining shear strength V_c, whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable-depth members may be included.

11.1.2 Maximum factored shear force V_u at supports may be computed in accordance with Article 11.1.2.1 or 11.1.2.2 when both of the following conditions are satisfied:

(a) Support reaction in correction of applied shear, introduces compression into the end regions of member, and

(b) No concentrated load occurs between face of support and location of critical section defined in Article 11.1.2.1 or 11.1.2.2

11.1.2.1 For non-pre-stressed members, sections located less than distance d from face of support may be designed for the same shear V_u as that computed at a distance d.

11.1.2.2 For pre-stressed members, sections located less than a distance h/2 from face of support may be designed for the same shear V_u as that computed at a distance h/2.

11.1.3 For deep flexural members, brackets and corbels, walls and slabs and footings, the special provisions of Article 11.8 through 11.11 shall apply.

11.2 Low density concrete

11.2.1 Provisions for shear strength v_c and torsional moment strength T_c apply to normal density concrete. When low-density aggregate concrete is used, one of the following modifications shall apply:

11.2.1.1 When f_{ct} is specfied and concrete is prportioned is accordance with Section 4.2, provisions for V_c and T_c shall be modified by substituting $1.8\ f'_{ct}$ for $\sqrt{f'_c}$, but the value of $1.8f_{ct}$ shall not exceed $\sqrt{f'_c}$.

11.2.1.2 When f_{ct} is not specified, all values of $\sqrt{f_c}$ affecting V_c, T_c, and M_{cr} shall be multiplied by 0.75 for "all low-density" concrete, and 0.85 for "sand-low-density" concrete. Linear interpolation may be used when partial sand replacement is used.

11.3 Shear strength provided by concrete for non-pre-stressed members

11.3.1 Shear strength, V_c shall be computed by provisions of Article 11.3.1.1. through 11.3.1.4 unless a more detailed calculation is made in accordance with Article 11.3.2.

11.3.1.1 For members subject to shear and flexure only,

$$V_c = (\sqrt{f'_c}/6)b_w d \qquad (11\text{-}3)$$

11.3.1.2 For members subject to axial compression,

$$V_c = \left(1 + \frac{N_u}{14A_g}\right)(\sqrt{f'_c}/6)b_w d \qquad (11\text{-}4)$$

Quantity N_u/A_g shall be expressed in MPa.

11.3.1.3 For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear.

11.3.1.4 At sections where factored torsional moment T_u exceeds $\phi\ (\sqrt{f'_c}/20)\ x^2y$

$$V_c = \frac{(\sqrt{f'_c}/6)b_w d}{\sqrt{1 + \left(2.5C_t\frac{T_u}{V_u}\right)^2}} \qquad (11\text{-}5)$$

11.3.2 Shear strength V_c may be computed by the more detailed calculation of Article 11.3.2.1 through 11.3.2.3

11.3.2.1 For members subject to shear and flexure only,

$$V_c = \left[\left(\sqrt{f'_c} + 120\rho_w\frac{V_u d}{M_u}\right) \div 7\right] b_w d \qquad (11\text{-}6)$$

but not greater than $0.3\sqrt{f'_c}b_w d$ quantity $V_u d/M_u$ shall not be taken greater than 1.0 in computing V_c by Eq. (11-6) where M_u is factored moment occurring simultaneously with V_u at section considered.

11.3.2.2 For members subject to axial compression, Eq. (11-6) may be used to compute V_c with M_m substituted for M_u and $V_u d/M_u$ not then limited to 1.0 where

$$M_m = M_u - N_u\frac{(4h - d)}{8} \qquad (11\text{-}7)$$

However, V_c shall not be taken greater than

$$V_c = 0.3\sqrt{f'_c}b_w d\sqrt{1 + \frac{0.3N_u}{A_g}} \qquad (11\text{-}8)$$

Quantity N_u/A_g shall be expressed in MPa. When M_m as computed by Eq. (11-7) is negative, V_c shall be computed by Eq. (11-8)

11.3.2.3 For members subject to significant axial tension,

$$V_c = \left(1 + \frac{0.3N_u}{A_g}\right)(\sqrt{f'_c}/6)b_w d \quad (11\text{-}9)$$

where N_u is negative for tension. Quantity N_u/A_g shall be expressed in MPa.

11.4 Shear strength provided by concrete for pre-stressed members

11.4.1 For members with effective pre-stress force not less than 40 percent of the tensile strength of flexural reinforcement, unless a more detailed calculation is made in accordance with Article 11.4.2.

$$V_c = \left(\frac{\sqrt{f'_c}}{20} + 5\frac{V_u d}{M_u}\right)b_w d \quad (11\text{-}10)$$

but V_c need not be taken less than $(\sqrt{f'_c}/6)\,b_w d$ nor shall V_c be taken greater than $0.4\sqrt{f'_c}\,b_w d$ nor the value given in Article 11.4.3 or 11.4.4. The quantity $V_u d/M_u$ shall not be taken greater than 1.0 where M_u is factored moment occurring simultaneously with V_u at section considered. When applying Eq. (11-10), d in the term $V_u d/M_u$ shall be the distance from extreme compression fibre to centroid of pre-stressed reinforcement.

11.4.2 Shear strength V_c may be computed in accordance with Articles 11.4.2.1 and 11.4.2.2, where V_c shall be the lesser of V_{ci} or V_{cw}.

11.4.2.1 Shear strength V_{ci} shall be computed by

$$V_{ci} = (\sqrt{f'_c}/20)b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \quad (11\text{-}11)$$

but V_{ci} need not be taken less than $(\sqrt{f'_c}/7)b_w d$, where

$$M_{cr} = (I/y_t)\,[(\sqrt{f'_c}/2) + f_{pe} - f_d] \quad (11\text{-}12)$$

and values of M_{max} and V_i shall be computed from the load combination causing maximum moment to occur at the section.

11.4.2.2 Shear strength V_{cw} shall be computed by

$$V_{cw} = 0.3(\sqrt{f'_c} + f_{pc})b_w d + V_p$$

(11-13)

Alternatively, V_{cw} may be computed as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $\sqrt{f'_c}/3$ at centroidal axis of member, or at intersection of flange and web when centroidal axis is in the flange and web when centroidal axis is in the flange. In composite members, principal tensile stress shall be computed using the cross-section that resists live load.

11.4.2.3 In Eq. (11-11) and (11-13) d shall be the distance from extreme compression fibre to centroid of pre-stressed reinforcement or 0.08h, whichever is greater.

11.4.3 In a pre-stressed member in which the section at a distance h/2 from face of support is closer to end of member than the transfer length of the pre-stressing tendons, the reduced pre-stress shall be considered when computing V_{cw}. This value of V_{cw} shall also be taken as the maximum limit for Eq. (11-10). Pre-stress force may be assumed to vary linearly from zero at end of tendon to a maximum at a distance from end of tendon equal to the transfer length, assuming to be 50 diameters for strand and 100 diameters for single wire.

11.4.4 In a pretensioned member where bonding of some tendons does not extend to end of member, a reduced pre-stress shall be considered when computing V_c in accordance with Articles 11.4.1 or 11.4.2. Value of V_{cw} calculated using the reduced pre-stress shall also be taken as the maximum limit for Eq. (11-10). Pre-stress force due to tendons for which bonding does not extend to end of member, may be assumed to vary linearly from zero at the point at which bonding commences to a maximum at a distance from this point equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

11.5 Shear strength provided by shear reinforcement

11.5.1 TYPES OF SHEAR REINFORCEMENT

11.5.1.1 Shear reinforcement may consist of:

(a) Stirrups perpendicular to axis of member
(b) Welded wire fabric with wires located perpendicular to axis of member.

11.5.1.2 For non-pre-stressed members, shear reinforcement may also consist of:

(a) Stirrups making an angle of 45 deg or more with longitudinal tension reinforcement
(b) Longitudinal reinforcement with bent portion making an angle of 30 deg or more with the longitudinal tension reinforcement
(c) Combinations of stirrups and bent longitudinal reinforcement
(d) Spirals

11.5.2 Design yield strength of shear reinforcement shall not exceed 400 MPa.

11.5.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fibre and shall be anchored at both ends according to Article 12.13 to develop the design yield strength of reinforcement.

11.5.4 SPACING LIMITS FOR SHEAR REINFORCEMENT

11.5.4.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed d/2 in non-pre-stressed members and (3/4)h in pre-stressed members, nor 600 mm.

11.5.4.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45 deg line, extending toward the reaction from mid-depth of member d/2 to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

11.5.4.3 When V_s exceeds $(\sqrt{f'_c}/3)b_w d$, maximum spacings given in Articles 11.5.4.1 and 11.5.4.2 shall be reduced by one-half.

11.5.5 MINIMUM SHEAR REINFORCEMENT

11.5.5.1 A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members (pre-stressed and non-pre-stressed) where factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c, except:

(a) Slabs and footings

(b) Concrete joist construction defined by Article 8.11

(c) Beams with total depth not greater than 250 mm, 2 1/2 times thickness of flange, or 1/2 the width of web, whichever is the greatest.

11.5.5.2 Minimum shear reinforcement requirements of Article 11.5.5.1 may be waived if shown by tests that required nominal flexural and shear strengths can be developed when shear reinforcement is omitted. Such tests shall simulate effects of differential settlement, creep, shrinkage and temperature change, based on a realistic assessment of such effects occurring in service.

11.5.5.3 Where shear reinforcement is required by Article 11.5.5.1 or by analysis, and where factored torsional moment T_u does not exceed $\phi(\sqrt{f'_c}/20 \sum x^2y)$, minimum area of sheer reinforcement for pre-stressed (except as provided in Article 11.5.5.4) and non-pre-stressed members shall be computed by

$$A_v = \frac{b_w S}{3f_y} \tag{11-14}$$

11.5.5.4 For pre-stressed members with effective pre-stress force not less than 40 percent of the tensile strength of flexural reinforcement, minimum area of shear reinforcement may be computed by Eq. (11-14) or (11-15).

$$A_v = \frac{A_{ps}}{80}\frac{f_{pu}}{f_y}\frac{s}{d}\sqrt{\frac{d}{b_w}} \tag{11-15}$$

11.5.5.5 Where factored torsional moment T_u exceeds $\phi\ (\sqrt{f'c}/20)\sum x^2y$, and where web reinforcement is required by Article 11.5.5.1 or by analysis, minimum area of closed stirrups shall be computed by

$$A_v + 2A_t = \frac{b_w s}{3f_y} \tag{11-16}$$

11.5.6 DESIGN OF SHEAR REINFORCEMENT

11.5.6.1 Where factored shear strength force V_u exceeds shear strength $ØV_c$, shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2) where shear strength V_s shall be computed in accordance with Articles 11.5.6.2 through 11.5.6.8.

11.5.6.2 When shear reinforcement perpendicular to axis of member is used,

$$V_s = \frac{A_v f_y d}{s} \qquad (11\text{-}17)$$

11.5.6.3 When inclined stirrups are used as shear reinforcement,

$$V_s = \frac{A_v f_y (\sin\alpha + \cos\alpha) d}{s} \qquad (11\text{-}18)$$

11.5.6.4 When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support,

$$V_s = A_v f_y \sin\alpha \qquad (11\text{-}19)$$

but not greater than $(\sqrt{f'_c}/4)b_w d$.

11.5.6.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, shear strength Vs shall be computed by Eq. (11-18)

11.5.6.6 Only the centre three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

11.5.6.7 Where more than one type of shear reinforcement is used to reinforce the same portion of a member, shear strength V_s shall be computed as the sum of the V_s values computed for the various types.

11.5.6.8 Shear strength V_s shall not be taken greater than $(2\sqrt{f'_c}/3)b_w d$.

11.6 Combined shear and torsion strength for non-pre-stressed members with rectangular or flanged sections

11.6.1 Torsion effects shall be included with shear and flexure where factored torsional moment T_u exceeds $\phi\left[(\sqrt{f'_c}/20)\sum x^2 y\right]$. Otherwise, torsion effects may be neglected.

11.6.1.1 For members with rectangular or flanged sections, the sum shall be taken for the component rectangles of the section, but the overhanging flange width used in design shall not exceed 3 times the flange thickness.

11.6.1.2 A rectangular box section may be taken as a solid section provided wall thickness h is at least x/4. A box section with wall thickness less than x/4, but greater than x/10, may also be taken as a solid section except that x^2y shall be multiplied by 4h/x. When h is less than x/10, stiffness of wall shall be considered. Fillets shall be provided at interior corners of all box sections.

11.6.2 If the factored torsional moment T_u in a member is required to maintain equilibrium, the member shall be designed to carry that torsional moment in accordance with Articles 11.6.4 through 11.6.9.

11.6.3 In a statically indeterminate structure where reduction of torsional moment in a member can occur due to distribution of internal forces, maximum factored torsional moment, T_u may be reduced to $\phi\left[(\sqrt{f'_c}/9)\sum x^2y/3\right]$.

11.6.3.1 In such a case the correspondingly adjusted moments and shears in adjoining members shall be used in design.

11.6.3.2 In lieu of more exact analysis, torsional loading from a slab shall be taken as uniformly distributed along a member.

11.6.4 Articles located less than a distance d from face of support may be designed for the same torsional moment T_u as that computed at a distance d.

11.6.5 TORSIONAL MOMENT STRENGTH

Design of cross sections subject to torsion shall be based on

$$T_u \leq \phi T_n \tag{11-20}$$

where T_u is factored torsional moment at section considered and T_n is nominal torsional moment strength computed by

$$T_n = T_c + T_s \tag{11-21}$$

where T_c is nominal torsional moment strength provided by concrete in accordance with Article 11.6.6, and T_s is nominal torsional moment strength provided by torsion reinforcement in accordance with Article 11.6.9.

11.6.6 TORSIONAL MOMENT STRENGTH PROVIDED BY CONCRETE

11.6.6.1 Torsional moment strength T_c shall be computed by

$$T_c = \frac{(\sqrt{f'_c}/15)\Sigma x^2 y}{\sqrt{1 + \left(\frac{0.4 V_u}{C_t T_u}\right)^2}} \qquad (11\text{-}22)$$

11.6.6.2 For members subject to significant axial tension, torsion reinforcement shall be designed to carry the total torsional moment, unless a more detailed calculation is made in which T_c given by Eq. (11-22) and V_c given by Eq. (11-5) shall be multiplied by $(1 + 0.3N_u/A_g)$, where N_u is negative for tension.

11.6.7 TORSION REINFORCEMENT REQUIREMENTS

11.6.7.1 Torsion reinforcement, where required, shall be provided in addition to reinforcement required to resist shear, flexure and axial forces.

11.6.7.2 Reinforcement required for torsion may be combined with that required for other forces, provided the area furnished is the sum of individually required areas and the most restrictive requirements for spacing and placement are met.

11.6.7.3 Torsion reinforcemnt shall consist of closed stirrups, closed ties, or spirals, combined with longitudinal bars.

11.6.7.4 Design yield strength of torsion reinforcement shall not exceed 400 MPs.

11.6.7.5 Stirrups and other bars and wires used for torsion reinforcement shall extend to a distance d from extreme compression fibre and shall be anchored according to Article 12.14 to develop the design yield strength of reinforcement.

11.6.7.6 Torsion reinforcement shall be provided at least a distance $(b_t + d)$ beyond the point theoretically required.

11.6.8 SPACING LIMITS FOR TORSION REINFOREMENT

11.6.8.1 Spacing of closed stirrups shall not exceed the smaller of $(x_1 + y_1)/4$, or 300 mm.

11.6.8.2 Spacing of longitudinal bars, not less than no. 10 distributed around the perimeter of the closed stirrups, shall not exceed 300 mm. At least one longitudinal bar shall be placed in each corner of the closed stirrups.

11.6.9 DESIGN OF TORSION REINFOREMENT

11.6.9.1 Where factored torsional moment T_u exceeds torsional moment strength ϕT_c, torsion reinforcement shall be provided to satisfy Eq. (11-20) and (11-21) where torsional moment strength T_s shall be computed by

$$T_s = \frac{A_t \alpha_t x_1 y_1 f_y}{s} \qquad (11\text{-}23)$$

where A_t is the area of one leg of a closed stirrup resisting torsion within a distance s, and $\alpha_t = (2 + y_1/x_1)/3$ but not more than 1.50. Longitudinal bars distributed around the perimeter of the closed stirrups A_t shall be provided in accordance with Article 11.6.9.3.

11.6.9.2 A minimum area of closed stirrups shall be provided in accordance with Article 11.5.5.5.

11.6.9.3 Required area of longitudinal bars A_l distributed around the perimeter of the closed stirrups A_t shall be computed by

$$A_\ell = 2A_t \left(\frac{x_1 + y_1}{s}\right) \qquad (11\text{-}24)$$

or by

$$A_\ell = \left[\frac{2.8xs}{f_y}\left(\frac{T_u}{T_u + \dfrac{V_u}{3C_t}}\right) - 2A_t\right]\left(\frac{x_1 + y_1}{s}\right) \qquad (11\text{-}25)$$

whichever is greater. Value of A_l computed by Eq. (11-25) need not exceed that obtained by substituting

$$\frac{b_w s}{3f_y} \text{ for } 2A_t$$

11.6.9.4 Torsional moment strength T_s shall not exceed $4T_c$.

11.7 Shear friction

11.7.1 Provisions of Article 11.7 are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack,an interface between dissimilar materials, or an interface between two concretes cast at different times.

11.7.2 Design of cross-sections subject to shear transfer as described in Article 11.7.1 shall be based on Eq. (11-1), where V_n is calculated in accordance with provisions of Article 11.7.3 or 11.7.4.

11.7.3 A crack shall be assumed to occur along the shear plane considered. Required area of shear-friction reinforcement A_{vt} across the shear plane may be desinged using either Article 11.7.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

11.7.3.1 Provisions of Articles 11.7.5 through 11.7.10 shall apply for all calculations of shear transfer strength.

11.7.4 SHEAR-FRICTION DESIGN METHOD

11.7.4.1 When shear-friction reinforcement is perpendicular to shear plane, shear strength V_n shall be computed by

$$V_n = A_{vf} f_y \mu \tag{11-26}$$

where μ is coefficient of friction in accordance with Article 11.7.4.3.

11.7.4.2 When shear-friction reinforcement is inclined to shear plane, such that the shear force produces tension in shear-friction reinforcement, shear strength V_n shall be computed by

$$V_n = A_{vf} f_y (\mu \sin \alpha_f + \cos \alpha_f) \tag{11-27}$$

where α_f is angle between shear-friction reinforcement and shear plane.

11.7.4.3 Coefficient of friction μ in Eq. (11-26) and Eq. (11-27) shall be

Concrete placed monolithically	1.4λ
Concrete placed against hardened concrete with surface intentionally roughened as specified in Article 11.7.9	1.0λ
Concrete placed against hardened concrete not intentionally roughened	0.6λ
Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 11.7.10)	0.7λ

where λ = 1.0 for normal density concrete, 0.85 for "sand-low-density" concrete and 0.75 for "all-low-density" concrete. Linear interpolation may be applied when partial sand replacement is used.

11.7.5 Shear strength V_n shall not be taken greater than $0.2f'_cA_c$ nor $5.5A_c$ in newtons, where A_c is area of concrete section resisting shear transfer.

11.7.6 Design yield strength of shear-friction reinforcement shall not exceed 400 MPa.

11.7.7 Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vt}f_y$, when calculating required A_{vf}.

11.7.8 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

11.7.9 For the purpose of Article 11.7, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, interface shall be roughened to a full amplitude of approximately 5 mm.

11.7.10 When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

11.8 Special Provisions for Deep Flexural Members

11.8.1 Provisions of Article 11.8 shall apply for members with l_n/d less than 5 and loaded at top or compression face. See also Article 12.10.6.

11.8.2 Design of deep flexural members for shear shall be based on Eq. (11-1) and (11-2), where shear strength V_c shall be in accordance with Article 11.8.5 or 11.8.6, and shear strength V_s shall be in accordance with Article 11.8.7.

11.8.3 Shear strength V_n for deep flexural members shall not be taken greater than $(2\sqrt{f'_c}/3)b_wd$ when l_n/d is less than 2. When l_n/d is between 2 and 5,

$$V_n = \frac{2}{3}\left(10 + \frac{l_n}{d}\right)\sqrt{f'_c}b_wd \qquad (11\text{-}28)$$

11.8.4 Critical section for shear measured from face of support shall be taken at a distance 0.15 l_n for uniformly loaded beams and 0.50a for beams with concentrated loads, but not greater than d.

11.8.5 Unless a more detailed calculation is made in accordance with Article 11.8.6,

$$V_c = (\sqrt{f'_c}/6) b_w d \qquad (11\text{-}29)$$

11.8.6 Shear strength V_c may be computed by

$$V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left[\left(\sqrt{f'_c} + 120\rho_w \frac{V_u d}{M_u}\right) \div 7\right] b_w d \qquad (11\text{-}30)$$

except that the term

$$\left(3.5 - 2.5\frac{M_u}{V_u d}\right)$$

shall not exceed 2.5, and V_c shall not be taken greater than $(\sqrt{f'_c}/2) b_w d$. M_u is factored moment occurring simultaneously with V_u at the critical section defined in Article 11.8.4.

11.8.7 Where factored shear force V_u exceeds shear strength ϕV_c, shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed by

$$V_s = \left[\frac{A_v}{s}\left(\frac{1 + \frac{\ell_n}{d}}{12}\right) + \frac{A_{vh}}{s_2}\left(\frac{11 - \frac{\ell_n}{d}}{12}\right)\right] f_y d \qquad (11\text{-}31)$$

where A_v is area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s, and A_{vh} is area of shear reinforcement parallel to flexural reinforcement within a distance s_2.

11.8.8 Area of shear reinforcement A_v shall not be less than 0.0015 $b_w s$, and s shall not exceed d/5, nor 500 mm.

11.8.9 Area of shear reinforcement A_{vh} shall not be less than 0.0025 $b_w s_2$, and s_2 shal not exceed d/3, nor 500 mm.

11.8.10 Shear reinforcement required at the critical section defined in Article 11.8.4 shall be used throughout the span.

11.9 Special Provisions for Brackets and Corbels

11.9.1 Provisions of Article 11.9 shall apply to brackets and corbels with a shear span-to-depth ratio a/d not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u. Distance d shall be measured at face of support.

11.9.2 Depth at outside edge of bearing area shall not be less than 0.5d.

11.9.3 Article at face of support shall be designed to resist simultaneously a shear V_u, a moment $(V_u a + N_{uc}(h - d))$, and a horizontal tensile force N_{uc}.

11.9.3.1 In all design calculations in accordance with Article 11.9, strength reduction factor ϕ shall be taken equal to 0.85.

11.9.3.2 Design of shear-friction reinforcement A_{vt} to resist shear V_u shall be in accordance with Article 11.7.

11.9.3.2.1 For normal density concrete, shear strength V_n shall not be taken greater than $0.2 f'_c b_w d$ nor $5.5 b_w d$ in Newtons.

11.9.3.2.2 For "all-low-density" or "sand-low-density" concrete, shear strength V_n shall not be taken greater than $(0.2 - 0.07 a/d) f'_c b_w d$ nor $(5.5 - 1.9 a/d) b_w d$ in Newtons.

11.9.3.3 Reinforcement A_f to resist moment $(V_u a + N_{uc}(h - d)$ shall be computed in accordance with Articles 10.2 and 10.3.

11.9.3.4 Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \leq \phi A_n f_y$. Tensile force N_{uc} shall not be taken less than $0.2 V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc} shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.

11.9.3.5 Area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or $(2A_{vf}/3 + A_n)$.

11.9.4 Closed stirrups or ties parallel to A_s, with a total area A_h not less than $0.5 (A_s - A_n)$ shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

11.9.5 Ratio $\rho = A_s/bd$ shall not be less than $0.04(f'_c/f_y)$.

11.9.6 At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following: (a) by a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars; (b) by bending primary tension bars A_s back to form a horizontal loop, or (c) by some other means of positive anchorage.

11.9.7 Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).

11.10 Special Provisions for Walls

11.10.1 Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in Article 11.11. Design for horizontal shear forces in plane of wall shall be in accordance with Articles 11.10.2 through 11.10.8.

11.10.2 Design of horizontal section for shear in plane of wall shall be based on Eq. (11-1) and (11-2), where shear strength V_c shall be in accordance with Article 11.10.5 or 11.10.6 and shear strength V_s shall be in accordance with Article 11.10.9.

11.10.3 Shear strength V_n at any horizontal section for shear in plane of wall shall not be taken greater than $(5\sqrt{f'_c}/6)hd$.

11.10.4 For design for horizontal shear forces in plane of wall, d shall be taken equal to 0.8 l_w. A larger value of d, equal to the distance from extreme compression fibre to centre of force of all reinforcement in tension, may be used when determined by a strain compatibility analysis.

11.10.5 Unless a more detailed calculation is made in accordance with Article 11.10.6, shear strength V_c shall not be taken greater than $(\sqrt{f'_c}/6)$ hd for walls subject to N_u in compression, or V_c shall not be taken greater than the value given in Article 11.3.2.3 for walls subject to N_u in tension.

11.10.6 Shear strength V_c may be computed by Eq. (11-32) and (11-33), where V_c shall be the lesser of Eq. (11-32) or (11-33).

$$V_c = (\sqrt{f'_c}/4)hd + \frac{N_u d}{4\ell_w} \qquad (11\text{-}32)$$

or

$$V_c = \left\{\left[\sqrt{f'_c}/2 + \frac{\ell_w\left(\sqrt{f'_c} + 2\dfrac{N_u}{\ell_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}}\right] \div 10\right\} hd \qquad (11\text{-}33)$$

where N_u is negative for tension. When $(M_u/V_u - l_w/2)$ is negative, Eq. (11-33) shall not apply.

11.10.7 Articles located closer to wall base than a distance $l_w/2$ or one-half the wall height, whichever is less, may be designed for the same V_c as that computed at a distance $l_w/2$ or one-half the height.

11.10.8 When factored shear force V_u is less than $V_c/2$, reinforcement shall be provided in accordance with Article 11.10.9 or in accordance with Chapter 14. When V_u exceeds $\phi V_c/2$, wall reinforcement for resisting shear shall be provided in accordance with Article 11.10.9.

11.10.9 DESIGN FOR SHEAR REINFORCEMENT FOR WALLS

11.10.9.1 Where factored shear force V_u exceeds shear strength ϕV_c, horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s_2} \qquad (11\text{-}34)$$

where A_v is area of horizontal shear reinforcement within a distance s_2 and distance d is in accordance with Article 11.10.4. Vertical shear reinforcement shall be provided in accordance with Article 11.10.9.4.

11.10.9.2 Ratio ρ_h or horizontal shear reinforcement area to gross concrete area of vertical section shall not be less than 0.0025.

11.10.9.3 Spacing of horizontal shear reinforcement s_2 shall not exceed $l_w/5$, 3h nor 500 mm.

11.10.9.4 Ratio $_n$ of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_h - 0.0025) \qquad (11\text{-}35)$$

nor 0.0025, but need not be greater than the required horizontal shear reinforcement.

11.10.9.5 Spacing of vertical shear reinforcement s_1 shall not exceed $l_w/3$, 3h nor 500 mm.

11.11 Special Provisions for Slabs and Footings

11.11.1 Shear strength of slabs and footings in the vicinity of concentrated loads or reactions is governed by the more severe of two conditions:

11.11.1.1 Beam action for slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 11.1 through 11.5.

11.11.1.2 Two-way action for slab or footing, with a critical section perpendicular to plane of slab and located so that its perimeter b_o is a minimum, but need not approach closer than d/2 to perimeter of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 11.11.2 through 11.11.4.

11.11.2 Design of slab or footing for two-way action shall be based on Eq. (11-1), where shear strength V_n shall not be taken greater than shear strength V_c computed in accordance with Article 11.11.2.1 or 11.11.2.2 unless shear reinforcement is provided in accordance with Article 11.11.3 or 11.11.4.

11.11.2.1 For non-pre-stressed slabs and footings:

$$V_c = (1 + 2/\beta_c)(\sqrt{f'_c}/6)b_o d \qquad (11\text{-}36)$$

but not greater than (f'_c,/3)$b_o d$. β_c is the ratio of long side to short side of concentrated. Load or reaction area and b_o is perimeter of critical section defined in Article 11.11.1.2.

11.11.2.2 At columns of two-way pre-stressed slabs and footings that meet requirements of Article 18.9.3:

$$V_c = 0.3(\sqrt{f'_c} + f_{pc})b_o d + V_p \qquad (11\text{-}37)$$

where b_o is perimeter of critical section defined in Article 11.11.1.2, f_{pc} is average value of f_{pc} for the two directions, and V_p is vertical component of all effective pre-stress forces crossing the critical section. If shear strength is computed by Eq. (11-37), the following shall be satisifed; otherwise, Eq. (11-36) shall apply:

(a) no portion of columns cross-section shall be closer to a discontinuous edge than 4 times the slab thickness, and

(b) f'_c in Eq. (11-37) shall not be taken greater than 35 MPa, and

(c) f_{pc} in each direction shall not be less than 0.9 MPa, nor be taken greater than 3.5 MPa.

11.11.3 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following:

11.11.3.1 Shear strength V_n shall be computed by Eq. (11-2), where shear strength V_c shall be in accordance with Article 11.11.3.4, and shear strength V_s shall be in accordance with Article 11.11.3.5.

11.11.3.2 Shear strength V_n shall not be taken greater than $(\sqrt{f'_c}/2)b_o d$, where b_o is perimeter of critical section defined in Article 11.11.3.3.

11.11.3.3 Shear strength shall be investigated at the critical section defined in Article 11.11.1.2 and at successive sections more distant from the support.

11.11.3.4 Shear strength V_c at any section shall not be taken greater than $(\sqrt{f'_c}/6)b_o d$, where b_o is perimeter of critical section defined in Article 11.11.3.3.

11.11.3.5 Where factored shear force V_u exceeds shear strength ϕV_c as given in Article 11.11.3.4, required area A_v and shear strength V_s of shear reinforcement shall be calculated in accordance with Article 11.5 and anchored in accordance with Article 12.13.

11.11.4 Shear reinforcement consisting of steel I- or channel-shaped sections (shearheads) may be used in slabs. Provisions of Articles 11.11.4.1 through 11.11.4.9 shall apply where shear due to gravity load is transferred at interior column supports. Where moment is transferred to columns, Article 11.12.2.5 shall apply.

11.11.4.1 Each shearhead shall consist of steel shapes fabricated by welding with a full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section.

11.11.4.2 Shearhead shall not be deeper than 70 times the web thickness of the steel shape.

11.11.4.3 Ends of each shearhead arm may be cut at angles not less than 30 deg with the horizontal, provided the plastic moment strength of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead.

11.11.4.4 All compression flanges of steel shapes shall be located within 0.3d of compression surface of slab.

11.11.4.5 Ratio α_v between the stiffness for each shearhead arm and that for surrounding composite cracked slab section of width $(c_2 + d)$ shall not be less than 0.15.

11.11.4.6 Plastic moment strength M_p required for each arm of the shearhead shall be computed by

$$\phi M_p = \frac{V_u}{2\eta}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \qquad (11\text{-}38)$$

where ϕ is strength reduction factor for flexure, is number of arms, and l_v is minimum length of each shearhead arm required to comply with requirements of Articles 11.11.4.7 and 11.11.4.8.

11.11.4.7 Critical slab section for shear shall be perpendicular to plane of slab and shall cross each shearhead arm three-quarters of the distance $[lv - (c_1/2)]$ from column face to end of shearhead arm. Critical section shall be located so that its perimeter b_o is a minimum, but need not approach closer than d/2 to perimeter of column section.

11.11.4.8 Shear strength V_n shall not be taken greater than $(\sqrt{f'_c}/3)b_o d$, on the critical section defined in Article 11.11.4.7. When shearhead reinforcement is provided, shear strength V_n shall not be taken greater than $0.6\sqrt{f'_c}b_o d$ on the critical section defined in Article 11.11.1.2.

11.11.4.9 A shearhead may be assumed to contribute a moment resistance M_v to each slab column strip computed by

$$M_v = \frac{\phi\alpha_v V_u}{2\eta}\left(\ell_v - \frac{c_1}{2}\right) \qquad (11\text{-}39)$$

where ϕ is the strength reduction factor for flexure, η is the number of arms, and l_v is the length of each shearhead arm actually provided. However, Mv shall not be taken larger than the smaller of:

(a) 30 percent of total factored moment required for each slab column strip,

(b) change in column strip moment over length l_v,

(c) value of M_p computed by Eq. (11-38).

11.11.4.10 When unbalanced moments are considered, shearhead must have adequate anchorage to transmit M_p to column.

11.11.5 OPENINGS IN SLABS

When openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in Chapter 13, the critical slab section for shear defined in Articles 11.11.1.2 and 11.11.4.7 shall be modified as follows:

11.11.5.1 For slabs without shearheads, that part of the perimeter of the critical section that is enclosed by straight lines projecting from the centroid of the load or reaction area and tangent to the boundaries of the openings shall be considered ineffective.

11.11.5.2 For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in Article 11.11.5.1.

11.12 Transfer of Moments to Columns

11.12.1 GENERAL

11.12.1.1 When gravity load, wind earthquake, or other lateral forces cause transfer of moment between slab and columns, a fraction of the unbalanced moment shall be transferred by eccentricity of shear in accordance with Articles 11.12.2.3 through 11.12.2.5.

11.12.2.2. Fraction of unbalanced moment not transferred by eccentricity of shear shall be transferred by flexure in accordance with Article 13.3.3.

11.12.2.3 A fraction of the unbalanced moment given by

$$\gamma_v = 1 - \frac{1}{1 + 2/3\sqrt{\dfrac{c_1 - d}{c_2 + d}}} \qquad (11\text{-}40)$$

shall be considered transferred by eccentricity of shear about centroid of a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than d/2 to perimeter of column.

11.12.2.4 Shear stresses resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly about centroid of critical section defined in Article 11.12.2.3. Maximum shear stress due to factored shear forces and moments shall not exceed v_c computed in accordance with Article 11.12.2.4.1 or 11.12.2.4.2.

11.12.2.4.1 For non-pre-stressed slabs:

$$v_c = \phi(1 + 2/\beta_c)\sqrt{f'_c}/6 \qquad (11\text{-}41)$$

but not greater than $\phi\sqrt{f'_c}/3$.

11.12.2.4.2 At columns of two-way pre-stressed slabs that meet requirements of Article 18.9.3:

$$v_c = \phi(0.3\sqrt{f'_c} + 0.3f_{pc} + V_p/b_o d) \qquad (11\text{-}42)$$

where b_o is perimeter of critical section defined in Article 11.12.2.3 and V_p is vertical component of all effective pre-stress forces crossing the critical section. If permissible shear stress is computed by Eq. (11-42), the following shall be satisified; otherwise, Eq. (11-41) shall apply.

(a) no portion of column cross section shall be closer to a discontinuous edge than four times the slab thickness, and

(b) f'_c in Eq. (11-42) shall not be taken greater than 35 MPa, and

(c) f_{pc} in each direction shall not be less than 0.9 MPa, nor be taken greater than 3.5 MPa.

11.12.2.5 When shear reinforcement consisting of steel I- or channel-shaped sections (shearheads) is provided, the sum of shear stresses due to vertical load acting on the critical section defined by Article 11.11.4.7 and moment transferred by eccentricity of shear about centroid of the critical section defined in Article 11.11.1.2 shall not exceed $\phi\sqrt{f'_c}/3$.

SUB-SECTION 12 DEVELOPMENT AND SPLICES OF REINFORCEMENT

12.1 Development of Reinforcement - General

Calculated tension or compression in reinforcement at each section of reinforced concrete members shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination therof. Hooks may be used in developing bars in tension only.

12.2 Development of deformed bars and deformed wire in tension

12.2.1 Development length l_d, in millimeters, for deformed bars and deformed wire in tension shall be computed as the product of the basic development length l_{db} of Article 12.2.2 and the applicable modification factor or factors of Articles 12.2.3 and 12.2.4, but l_d shall not be less than that specified in Article 12.2.5.

12.2.2 Basic development length l_{db} shall be:

No. 35 bar and smaller	$0.02A_b f_y/\sqrt{f'c}$*
but not less than	$0.06\ d_b f_y$**
No. 45 bar	$25f_y/\sqrt{f'_c}$***
No. 55 bar	$35\ f_y/\sqrt{f'_c}$***
Deformed wire	$(3d_b f_y/8)\sqrt{f'_c}$

12.2.3 Basic development length l_{db} shall be multiplied by applicable factor or factors for:

12.2.3.1 Top reinforcements**** 1.4

12.2.3.2 Yield strength

Reinforcement with f_y greater than 400 MPa $\quad 2 - \dfrac{400}{f_y}$

---

* The constant carries the unit of one/mm

** The constant carries the unit of mm^2/N

*** The constant carries the unit of mm

**** Top reinforcement is horizontal reinforcement so placed that more than 300 mm of fresh concrete is cast in the member below the reinforcement

12.2.3.3 Low-density aggregate concrete

When f_{ct} is specified and concrete is proportioned in accordance with Article 4.2 $\sqrt{f'_c}/1.8f_{ct}$ but not less than 1.0

When f_{ct} is not specified:
"all-low-density" concrete 1.33
"sand-low-density" concrete 1.18
Linear interpolation may be applied when partial sand replacement is used.

12.2.4 Basic development length l_{db} is modified by appropriate factors of Article 12.2.3 may be multiplied by applicable factor or factors for:

12.2.4.1 Bar spacing

Reinforcement being developed in length under consideration and spaced laterally at least 150 mm on centre with at least 70 mm clear from face of member to edge bar, measured in direction of spacing. 0.8

12.2.4.2 Excess reinforcement

Where anchorage or development for f_y is not specifically required, reinforcement in flexural member in excess of that required by analysis . . . .

.... (A_s required)/(A_s provided)

12.2.4.3 Spirals

Reinforcement enclosed within spiral reinforcement not less than 5 mm diameter and not more than 100 mm pitch 0.75

12.2.5 Development length l_d shall not be less than 300 mm except in computation of lap splices by Article 12.15 and development of web reinforcement by Article 12.13.

12.3 Development of deformed bars in compression

12.3.1 Development length l_d, in millimeters, for deformed bars in compression shall be computed as the product of the basic development length l_{db} of Article 12.3.2 and applicable modifcation factors of Article 12.3.3 but l_d shall not be less than 200 mm.

12.3.2 Basic development length
l_{db} shall be.......................... $(d_b f_y/4)\sqrt{f'_c}$
but not less than..................... $0.04 d_b f_y$*

12.3.3 Basic development length l_{db} may be multiplied by applicable factors for:

12.3.3.1 Excess reinforcement

Reinforcement in excess of that
required by analysis..........(A_s required)/(As provided)

12.3.2.3 Spirals

Reinforcement enclosed within spiral
reinforcement not less than 5 mm
diameter and not more than 100 pitch......... 0.75

12.4 Development of bundled bars

Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20 percent for three-bar bundle, and 33 percent for four-bar bundle.

12.5 Development of standard hooks in tension

12.5.1 Development length l_{dh} in millimeters, for deformed bars in tension terminating in a standard hook (Article 7.1) shall be computed as the product of the basic development length l_{hb} of Article 12.5.2 and the applicable modification factor or factors of Article 12.5.3 but l_{dh} shall not be less than $8d_b$ or 150 mm, whichever is greater.

12.5.2 Basic development length l_{hb} for a
hooked bar with f_y equal to 400
MPa shall be .... $100 d_b/\sqrt{f'_c}$**

12.5.3 Basic development length l_{hb} shall be multiplied by applicable factor or factors for:

12.5.3.1 Bar yield strength

Bars with f_y other than 400 MPa .... $f_y/400$

12.5.3.2 Concrete cover

For no.35 bar and smaller, side cover
(normal to plane of hook) not less
than 60 mm and for 90 deg hook,
cover on bar extension beyond hook
not less than 50 mm 0.7

* The constant carries the unit of mm^2/N

** The constant carries the unit of N/mm^2

12.5.3.3 Ties or stirrups

For No. 35 bar or smaller, hook enclosed vertically or horizontally within ties or stirrup ties spaced along the full development length l_{dh} not greater than $3d_b$, where d_b is diameter of hooked bar........... 0.8

12.5.3.4 Excess reinforcement
Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis .... (A_s required)/(A_s provided)

12.5.3.5 Low density aggregate concrete 1.3

12.5.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 60 mm, hooked bar shall be enclosed within ties or stirrup ties spaced along the full development length l_{dh} not greater than $3d_b$ where d_b is diameter of hooked bar. For this case, factor of Article 12.5.3.3 shall not apply.

12.5.5 Hooks shall not be considered effective in developing bars in compression.

12.6 Mechanical storage

12.6.1 Any mechanical device capable of developing the strength of reinforcement without damage to concrete may be used as anchorage.

12.6.2 Test results showing adequacy of such mechanical devices shall be presented to the Building Official.

12.6.3 Development of reinforcement may consist of a combination of mechanical anchorage plus additional embedment strength of reinforcement between the point of maximum bar stress and the mechanical anchorage.

12.7 Development of welded deformed wire fabric in tension

12.7.1 Development length l_d. in millimeters, of welded deformed wire fabric measured from point of critical section to end of wire shall be computed as the product of the basic development length l_{db} of Article 12.7.2 or 12.7.3 and applicable modification factor or factors of Articles 12.2.3 and 12.2.4; but l_d shall not be less than 200 mm except in computation of lap splices by Article 12.18 and development of web reinforcement by Article 12.13.

12.7.2 Basic development length l_{db} of welded deformed wire fabric, with at least one cross wire within the development length not less than 50 mm from point of critical section, shall be

$$[3d_b(f_y - 140)/8]\sqrt{f'_c}^*$$

but not less than

$$2.5\frac{A_w}{s_w}\frac{f_y}{\sqrt{f'_c}}$$

12.7.3 Basic development length l_{db} of welded deformed wire fabric, with no cross wires within the development length, shall be determined as for deformed wire.

12.8 Development of welded smooth wire fabric in tension

Yield strength of welded smooth wire fabric shall be considered developed by embedment of two cross wires with the closer cross wire not less than 50 mm from point of critical section. However, basic development length l_{db} measured from point of critical section to outermost cross wire shall not be less than

$$3.3\frac{A_w}{s_w}\frac{f_y}{\sqrt{f'_c}}$$

modified by (A_s required)/(As provided) for reinforcement in excess of that required by analysis and by factor of Article 12.2.3 for low-density aggregate concrete, but l_d shall not be less than 150 mm except in computation of lap splices by Article 12.19.

12.9 Development of pre-stressing strand

12.9.1 Three-or seven-wire pre-tensioning strand shall be bonded beyond the critical section for a development length, in millimeters, not less than

$$\left(f_{ps} - \frac{2}{3}f_{se}\right)d_b/7\dagger$$

where d_b is strand diameter in millimeters, and f_{ps} and f_{se} are expressed in megapascals.

12.9.2 Investigation may be limited to cross sections nearest each end of the member that are required to develop full design strength under specified factored loads.

12.9.3 Where bonding of a strand does not extend to end of member, and design includes tension at service load in pre-compressed tensile zone as permitted by Article 18.4.2, development length specified in Article 12.9.1 shall be doubled.

12.10 Development of flexural reinforcement - General

12.10.1 Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforement on the opposite face of the member.

12.10.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. Provisions of Article 12.11.3 must be satisfied.

12.10.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of member or $12d_b$, whichever is greater, except at supports of simple spans and at free end of cantilevers.

12.10.4 Continuing reinforcement shall have an embedment length not less than the development length l_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

12.10.5 Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

12.10.5.1 Shear at the cut-off point does not exceed two-thirds that permitted, including shear strength of shear reinforement provided.

12.10.5.2 Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of member. Excess stirrup area A_v shall not be less than $0.4b_w s/f_y$. Spacing s shall not exceed $d/8\beta_b$ where β_b is the ratio of area of reinforcement cut off to total area of tension reinforcement at the section.

12.10.5.3 For No. 35 bar and smaller, continuing reinforcment provides double the area required for flexure at the cut off point and shear does not exceed three-fourths that permitted.

12.10.6 Adequate anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets, deep flexural members, or members in which tension reinforcement is not parallel to compression face.

12.11 Development of positive moment reinforcement

12.11.1 At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of member in the support. In beams, such reinforcement shall extend into the support at least 150 mm.

12.11.2 When a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by Article 12.11.1 shall be anchored to develop the specified yield stength f_y in tension at the face of support.

12.11.3 At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that l_d computed for f_y by Article 12.2 satisfies Eq. (12-1); except Eq. (12-1) need not be satisfied for reinforcement terminating beyond centre-line of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$\ell_d \leq \frac{M_n}{V_u} + \ell_a \qquad (12\text{-}1)$$

where,

M_n is nominal moment strength assuming all reinforcement at the section can be stressed to the specified yield strength f_y.

V_u is factored shear force at the section.

l_a at a support shall be the embedment length beyond centre of support.

l_a at a point of inflection shall be limited to the effective depth of member of 12 d_b, whichever is greater.

Value of M_n/V_u may be increased 30 percent when the ends of reinforcement are confined by a compressive reaction.

12.12 Development of negative moment reinforcement

12.12.1 Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks or mechanical anchorage.

12.12.2 Negative moment reinforcement shall have an embedment length into the span as required by Articles 12.1 and 12.10.3.

12.12.3 At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflections, not less than effective depth of member $12d_b$, or one-sixteenth the clear span, whichever is greater.

12.13 Development of web reinforement

12.13.1 Web reinforcement shall be carried as close to compression and tension surfaces of member as cover requirements and proximity of other reinforcements will permit.

12.13.2 Ends of single leg, simple U-, or multiple U-stirrups shall be anchored by one of the following means:

12.13.2.1 A standard hook plus an embedment of $0.5\ l_d$. The $0.5\ l_d$ embedment of stirrup leg shall be taken as the distance between mid-depth of member d/2 and the start of hook (point of tangency).

12.13.2.2 Embedment 2/d above or below middepth on the compression side of the member for a full development length l_d but not less than $24d_b$; or for deformed bars or deformed wire, 300 mm.

12.13.2.3 For no.15 bar and D31 wire, and smaller, bending around longitudinal reinforcement through at least 135 deg plus, for stirrups with design stress exceeding 300 MPa, an embedment of $0.33\ l_d$. The $0.33\ l_d$ embedment of a stirrup leg shall be taken as the distance between mid-depth of member d/2 and start of hook (point of tangency).

12.13.2.4 For each leg of welded smooth wire fabric forming simple U-stirrups, either:

(a) Two longitudinal wires spaced at 50 mm spacing along the member at the top of the U.

(b) One longitudinal wire located not more than d/4 from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first wire. The second wire may be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

12.13.2.5 For each leg of a single leg stirrup of welded smooth of deformed wire fabric, two longitudinal wires at a minimum spacing of 50 mm and with the inner wire at least the greater of d/4 or 50 mm from mid-depth of member d/2. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

12.13.3 Between anchored ends each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

12.13.4 Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall be continuous with longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond mid-depth d/2 as specified for development length in Article 12.2 for that part of f_y required to satisfy Eq. (11-19).

12.13.5 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are 1.7 l_d. In members at least 500 mm deep, such splices with $A_b f_y$ not more than 40 kN per leg may be considered adequate if stirrup legs extend the full available depth of member.

12.14 Splices of reinforcement - General

12.14.1 Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the Engineer.

12.14.2 LAP SPLICES

12.14.2.1 Lap splices shall not be used for bars larger than 35 except as provided in Articles 12.16.2 and 15.8.2.4.

12.14.2.2 Lap splices of bundled bars shall be based on the lap splice length required for individual bars within a bundle, increased 20 percent for a three-bar bundle and 33 percent for a four-bar bundle. Individual bar splices within a bundle shall not overlap.

12.14.2.3 Bars spliced by non-contact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, or 150 mm.

12.14.3 WELDED SPLICES AND MECHANICAL CONNECTIONS

12.14.3.1 Welded splices and other mechanical connections may be used.

12.14.3.2 Except as provided in this Code, all welding shall conform to "Structural Welding Code - Reinforcing Steel", (AWS D1.4)

12.14.3.3 A full welded splice shall have bars butted and welded to develop in tension at least 125 percent of specified yield strength f_y of the bar.

12.14.3.4 A full mechanical connection shall develop in tension or compression, as required, at least 125 percent of specified yield strength f_y of the bar.

12.14.3.5 Welded splices and mechanical connections not meeting requirements of Article 12.14.3.3 or 12.14.3.4 may be used in accordance with Article 12.15.4.

12.15 Splices of deformed bars and deformed wire in tension

12.15.1 Minimum length of lap for tension lap splices shall be required for Class A, B or C splice, but not less than 300 mm, where:

Class A splice . . . . 1.0 l_d

Class B splice . . . . 1.3 l_d

Class C splice . . . . 1.7 l_d

where l_d is the tensile development length for the specified yield strength f_y in accordance with Article 12.2.

12.15.2 Lap splices of deformed bars and deformed wire in tension shall conform to Table 12.15.

TABLE 12.15
TENSION LAP SPLICES

$\frac{A_s \text{ provided*}}{A_s \text{ required}}$	Maximum percent of A_s spliced within required lap length 50	75	100
Equal to or greater than 2	Class A	Class A	Class B
Less than 2	Class B	Class C	Class C

* Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location

12.15.3 Welded splices or mechanical connections used where area of reinforcement provided is less than twice that required by analysis shall meet requirements of Article 12.14.3.3 or 12.14.3.4.

12.15.4 Welded splices or mechanical connections used where area of reinforcement provided is at least twice that required by analysis shall meet the following:

12.15.4.1 Splices shall be staggered at least 600 mm and in such manner as to develop at every section at least twice the calculated tensile force at that section but not less than 140 MPa for total area of reinforcement provided.

12.15.4.2 In computing tensile force developed at each section, spliced reinforcement may be rated at the specified splice strength. Unspliced reinforcement shall be rated at that fraction of f_y defined by the ratio of the shorter actual development length of l_d required to develop the specified yield strength f_y.

12.15.5 Splices in "tension tie members" shall be made with a full welded splice or full mechanical connection in accordance with Article 12.14.3.3 or 12.14.3.4 and splices in adjacent bars shall be staggered at least 800 mm.

12.16 Splices of deformed bars in compression

12.16.1 Minimum length of lap for compression lap splices shall be the development length in compression computed in accordance with Article 1.3, but not less than 0.07 $f_y d_b$, nor $(0.13f_y - 24)d_b$ for f_y greater than 400 MPa, for 300 mm. For f'_c less than 20 MPa, length of lap shall be increased by one-third.

12.16.2 When bars of different size are lap spliced in compression, splice length shall be the larger of development length of larger bar, or splice length of smaller bar. Bar sizes No. 45 and No. 55 may be lap spliced to No. 35 and smaller bars.

12.16.3 In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than 0.0015 hs, lap splice length may be multiplied by 0.83, but lap length shall not be less than 300 mm. Tie legs perpendicular to dimension h shall be used in determining effective area.

12.16.4 In spirally reinforced compression members, lap splice length of bars within a spiral may be multiplied by 0.75, but lap length shall not be less than 300 mm.

12.16.5 Welded splices or mechanical connections used in compression shall meet requirements of Article 12.14.3.3 or 12.14.3.4.

12.16.6 END BEARING SPLICES

12.16.6.1 In bars required for compression only, compressive stress may be transmitted by bearing of square ends held in concentric contact by a suitable device.

12.16.6.2 Bar ends shall terminate in flat surfaces within 1-1/2 deg of a right angle to the axis of the bars and shall be fitted within 3 deg of full bearing after assembly.

12.16.6.3 End bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

12.17 Special splice requirements for columns

12.17.1 Where factored load stress in longitudinal bars in a column, calculated for various loading combinations, varies from f_y in compression to $1/2f_y$ or less in tension, lap splices, butt welded splices, mechanical connections, or end bearing splices may be used. Total tensile strength provided in each face of the column by splices alone or by splices in combinations with continuing unspliced bars at specified yield strength f_y shall be at least twice the calculated tension in that face of the column but not less than required by Article 12.17.3.

12.17.2 Where factored load stress in longitudinal bars in a column, calculated for any loading conbination, exceeds $1/2f_y$ in tension, lap splices designated to develop the specified yield strength f_y in tension, or full welded splices or full mechanical connections in accordance with Article 12.14.3.3 or 12.14.3.4 shall be used.

12.17.3 At horizontal cross-sections of columns where splices are located, a minimum tensile strength in each face of the column equal to one-quarter the area of vertical reinforcement in that face multiplied by f_y shall be provided.

12.18 Splices of welded deformed wire fabric in tension

12.18.1 Minimum length of lap for lap splices of welded deformed wire fabric measured between the ends of each fabric sheet shall be less not than 1.7 l_d nor 200 mm and the overlap measured between outermost cross-wires of each fabric sheet shall be not less than 50 mm. l_d shall be the development length for the specified yield strength f_y in accordance with Article 12.7.

12.18.2 Lap splices of welded deformed wire fabric, with no cross-wires within the lap splice length, shall be determined as for deformed wire.

12.19 Splices of welded smooth wire fabric in tension

Minimum length of lap for lap splices of welded smooth wire fabric shall be in accordance with the following:

12.19.1 When area of reinforcement provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross-wires of each fabric sheet shall be not less than one spacing of cross-wires plus 50 mm, not less than 1.5 l_d, nor 150 mm. l_d shall be the development length for the specified yield strength f_y, in accordance with Article 12.8.

12.19.2 When area of reinforcement provided is at least twice that required by analysis at splice location, length of overlap measured between outermost cross-wires of each fabric sheet shall not be less than 1.5 l_d, or 50 mm. l_d shall be the development length for the specified yield strength f_y in accordance with Article 12.8.

PART 2

STRUCTURAL DESIGN REQUIREMENTS

SECTION 6E

REINFORCED AND PRE-STRESSED CONCRETE

STRUCTURAL SYSTEMS OR ELEMENTS

PART 7
SECTION 6E

SUB-SECTION 5 TWO-WAY SLAB SYSTEMS

13.1 Scope

13.1.1 Provisions of Chapter 13 shall apply for design of slab systems reinforced for flexure in more than one direction with or without beams between supports.

13.1.2 A slab system may be supported on columns or walls. If supported by columns, no portion of a column capital or bracket shall be considered for structural purposes that lies outside the largest right circular cone, right pyramid, or tapered wedge whose planes are oriented no greater than 45 deg to the column.

13.1.3 Solid slabs and slabs with recesses or pockets made by permanent or removable fillers between ribs or joists in two directions are included within the scope of Chapter 13.

13.1.4 Minimum thickness of slabs designed in accordance with Chapter 13 shall be as required by Article 9.5.3.

13.2 Definitions

13.2.1 Column strip is a design strip with a width on each side of a column centerline equal to 0.25 l_2 or 0.25 l_1 whichever is less. Column strip includes beams, if any.

13.2.2 Middle strip is a design strip bounded by two column strips.

13.2.3 A panel is bounded by column, beam or wall centerlines on all sides.

13.2.4 For monolithic or fully composite construction, a beam includes that portion of slab on each side of the beam, extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

13.3 Design procedures

13.3.1 A slab system may be designed by any procedure satisfying conditions of equilibrium and geometric compatibility if shown that the design strength at every section is at least equal to the required strength considering Articles 9.2 and 9.3 and that all serviceability conditions, including specified limits on deflections are met.

13.3.1.1 For gravity loads, a slab system, including the slab and beams (if any) between supports and supporting columns or walls forming orthogonal frames, may be designed by either the Direct Design Method of Article 13.6 or the Equivalent Frame Method of Article 13.7.

13.3.1.2 For lateral loads, analysis of unbraced frames shall take into account effects of cracking and reinforcement on stiffness of frame members.

13.3.1.3 Results of the gravity load analysis may be combined with results of the lateral load analysis.

13.3.2 The slab and beams (if any) between supports shall be proportioned for factored moments prevailing at every section.

13.3.3 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, a fraction of the unbalanced moment shall be transferred by flexure in accordance with Articles 13.3.3.2 and 13.3.3.3.

13.3.3.1 Fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with Article 11.12.2.

13.3.3.2 A fraction of the unbalanced moment given by

$$\gamma_f = \frac{1}{1 + 2/3\sqrt{\dfrac{c_1 + d}{c_2 + d}}} \qquad (13\text{-}1)$$

shall be considered transferred by flexure over an effective slab width between lines that are one and one-half slab or drop panel thickness (1.5h) outside opposite faces of the column or capital.

13.3.3.3 Concentration of reinforcement over the column by closer spacing or additional reinforcement may be used to resist moment on the effective slab width defined in Article 13.3.3.2.

13.3.4 Design for transfer of load from slab to supporting columns or walls through shear and torsion shall be in accordance with Chapter 11.

13.4 Slab reinforcement

13.4.1 Areas of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections but shall not be less than required by Article 7.12.

13.4.2 Spacing of reinforcement at critical sections shall not exceed two times the slab thickness, except for portions of slab area that may be of cellular or ribbed construction. In the slab over cellular spaces, reinforcement construction shall be provided as required by Article 7.12.

13.4.3 Positive moment reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns or walls.

13.4.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, to be developed at face of support according to provisions of Chapter 12.

13.4.5 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement may be within the slab.

13.4.6 In slabs with beams between supports with a value of greater than 1.0, special top and bottom slab reinforcement shall be provided at exterior corners in accordance with the following:

13.4.6.1 The special reinforcement in both top and bottom of slab shall be sufficient to resist a moment equal to the maximum positive moment (per meter of width) in the slab.

13.4.6.2 Direction of moment shall be assumed parallel to the diagonal from the corner in the top of the slab and perpendicular to the diagonal in the bottom of the slab.

13.4.6.3 The special reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

13.4.6.4 In either the top or bottom of the slab, the special reinforcement may be placed in a single band in the direction of the moment or in two bands parallel to the sides of the slab.

13.4.7 Where a drop panel is used to reduce amount of negative moment reinforcement over the column of a flat slab, size of drop panel shall be in accordance with the following:

13.4.7.1 Drop panel shall extend in each direction from centreline of support a distance not less than one-sixth the span length measured from centre-to-centre of supports in that direction.

13.4.7.2 Projection of drop panel below the slab shall be at least one-quarter the slab thickness beyond the drop.

13.4.7.3 In computing required slab reinforcement, thickness of drop panel below the slab shall not be assumed greater than one-quarter the distance from edge of drop panel to edge of column of column capital.

13.4.8 DETAILS OF REINFORCEMENT IN SLABS WITHOUT BEAMS

13.4.8.1 In addition to the other requirements of Article 13.4, reinforcement in slabs without beams shall have minium bend point locations and extensions for reinforcement as prescribed in Fig. 13.4.8.

13.4.8.2 Where adjacent spans are unequal, extension of negative reinforcement beyond the face of support as prescribed in Fig. 13.4.8 shall be based on requirements of longer span.

13.4.8.3 Bent bars may be used only when depth-span ratio permits use of bends 45 deg or less.

13.4.8.4 For slabs in frames not braced against sidesway and for slabs resisting lateral loads, lengths of reinforcement shall be determined by anslysis but shall not be less than those prescribed in Fig. 13.4.8.

13.5 Openings in slab systems

13.5.1 Openings of any size may be provided in slab systems if shown by analysis that the design strength is at least equal to the required strength considering Articles 9.2 and 9.3 and that all serviceability conditions, including the specified limits on deflections, are met.

13.5.2 In lieu of special analysis as required by Article 13.5.1, openings may be provided in slab systems without beams only in accordance with the following:

13.5.2.1 Openings of any size may be located in the area common to intersecting middle strips, provided the total amount of reinforcement required for the panel without the opening is maintained.

13.5.2.2 In the area common to intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

13.5.2.3 In the area common to one column strip and one middle strip, not more than one-quarter the reinforcement in either strip shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

13.5.2.4 Shear requirements of Article 11.11.5 shall be satisfied.

13.6 Direct design method

13.6.1 LIMITATIONS

Slab systems within the following limitations may be designed by the Direct Design Method.

13.6.1.1 There shall be a minimum of three continuous spans in each direction.

13.6.1.2 Panels shall be rectangular with a ratio of longer to shorter span centre-to-cenre of supports within a panel not greater than 2.

13.6.1.3 Successive span lengths centre-to-centre of supports in each direction shall not differ by more than one-third the longer span.

13.6.1.4 Columns may be offset a maximum of 10 percent of the span (in direction of offset) from either axis between centrelines of successive columns.

13.6.1.5 All loads shall be due to gravity only and uniformly distributed over an entire panel. Live load shall not exceed three times dead load.

13.6.1.6 For a panel with beams between supports on all sides, the relative stiffness of beams in two perpendicular directions

$$\frac{\alpha_1 \ell_2^2}{\alpha_2 \ell_1^2} \qquad (13\text{-}2)$$

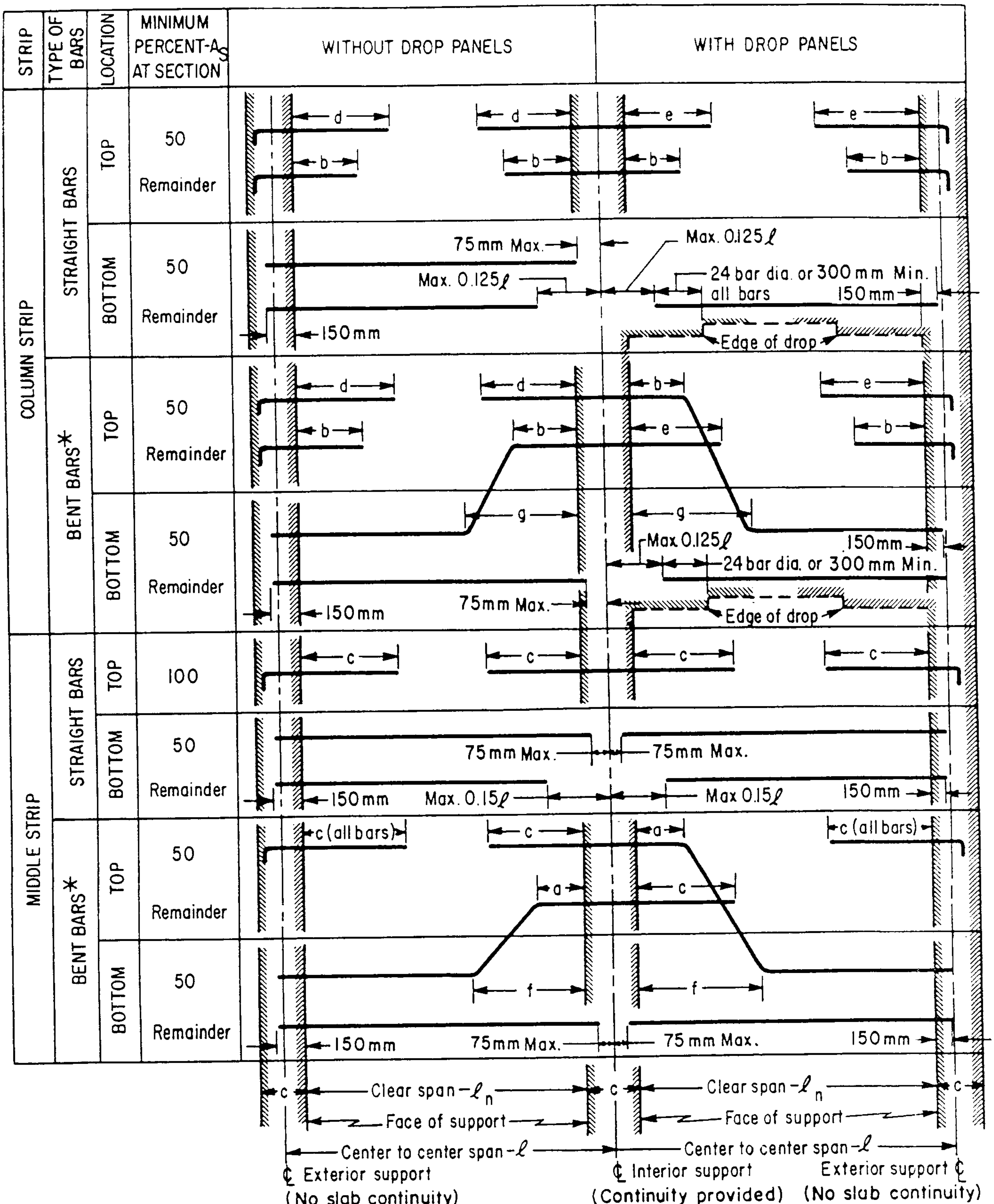
shall not be less than 0.2 nor greater than 5.0.

13.6.1.7 Moment redistribution as permitted by Article 8.4 shall not be applied for slab systems designed by the Direct Design Method. See Article 13.6.7

13.6.1.8 Variations from the limitations of Article 13.6.1 may be considered acceptable if demonstrated by analysis that requirements of Article 13.3.1 are satisfied.

13.6.2 TOTAL FACTORED STATIC MOMENT FOR A SPAN

13.6.2.1 Total factored static moment for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.



* Bent bars at exterior supports may be used if a general analysis is made

	BAR LENGTH FROM FACE OF SUPPORT						
	MINIMUM LENGTH					MAXIMUM LENGTH	
MARK	a	b	c	d	e	f	g
LENGTH	$0.14\ell_n$	$0.20\ell_n$	$0.22\ell_n$	$0.30\ell_n$	$0.33\ell_n$	$0.20\ell_n$	$0.24\ell_n$

Fig. 13.4.8 - Minimum bend point locations and extensions for reinforcement in slabs without beams (See Section 12.11.1 for reinforcement extension into supports)

13.6.2.2 Absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} \qquad (13\text{-}3)$$

13.6.2.3 Where the transverse span of panels on either side of the centerline of supports varies, l_2 in Eq. (13-3) shall be taken as the average of adjacent transverse spans.

13.6.2.4 When the span adjacent and parallel to an edge is being considered, the distance from the edge to panel centerline shall be substituted for l_2 in Eq. (13-3).

13.6.2.5 Clear span shall extend from face to face of columns, capitals, brackets, or walls. Value of used in Eq. (13-3) shall not be less than $0.65l_1$. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

13.6.3 NEGATIVE AND POSITIVE FACTORED MOMENTS

13.6.3.1 Negative factored moments shall be located at face of rectangular supports. Circular or regular polygon shaped supports shall be treated as square supports with the same area.

13.6.3.2 In an interior span, total static moment M_o shall be distributed as follows:

Negative factored moment. 0.65

Positive factored moment. 0.35

13.6.3.3 In an end span, total factored static moment M_o shall be distributed as follows:

	(1)	(2)	(3)	(4)	(5)
	Exterior edge unre-strained	Slab with beams between all supports	Slab without beams between interior supports: Without edge beam*	Slab without beams between interior supports: With edge beam	Exterior edge fully restrained
Interior negative factored moment	0.75	0.70	0.70	0.70	0.65
Positive factored moment	0.63	0.57	0.52	0.50	0.35
Exterior negative factored moment	0	0.16	0.26	0.30	0.65

* See article 13.6.5.6

13.6.3.4 Negative moment sections shall be designed to resist the larger of the two interior negative factored moments determined for spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with stiffnesses of adjoining elements.

13.6.3.5 Edge beams or edges of slab shall be proportioned to resist in torsion their share of exterior negative factored moments.

13.6.3.6 For moment transfer between slab and an edge column in accordance with Article 13.3.3.1, column strip nominal moment strength provided shall be used as the transfer moment for gravity load.

13.6.4 FACTORED MOMENTS IN COLUMN STRIPS

13.6.4.1 Column strips shall be proportioned to resist the following portions in percent of interior negative factored moments:

l_2/l_1	0.5	1.0	2.0
$(a_1 l_2/l_1) = 0$	75	75	75
$(a_1 l_2/l_1 > 1.0$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.2 Column strips shall be proportioned to resist the following portions in percent of exterior negative factored moments.

l_2/l_1		0.5	1.0	2.0
$(a_1 l_2/l_1) = 0$	$B_1 = 0$	100	100	100
	$B_1 > 2.5$	75	75	75
$(a_1 l_2/l_1) > 1.0$	$B_1 = 0$	100	100	100
	$B_1 > 2.5$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.3 Where supports consist of columns or walls extending for a distance equal to or greater than three-quarters the span length l_2 used to compute M_o, negative moments shall be considered to be uniformly distributed across l_2.

13.6.4.4 Column strips shall be proportioned to resist the following portions in percent of positive factored moments:

l_2/l_1	0.5	1.0	2.0
$(a_1 l_2/l_1) = 0$	60	60	60
$(a_1 l_2/l_1 > 1.0$	90	75	45

Linear interpolations shall be made between values shown.

13.6.4.5 For slabs with beams between supports, the slab portion of column strips shall be proportioned to resist that portion of column strip moments not resisted by beams.

13.6.5 FACTORED MOMENTS IN BEAMS

13.6.5.1 Beams between supports shall be proportioned to resist 85 percent of column strip moments if $(\alpha_1 l_2/l_1)$ is equal to or greater than 1.0.

13.6.5.2 Or values of $(\alpha_1 l_2/l_1)$ between 1.0 and zero, proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

13.6.5.3 In addition to moments calculated for uniform loads according to Articles 13.6.2.2. , 13.6.5.1, and 13.6.5.2, beams shall be proportioned to resist all moments caused by concentrated or linear loads applied directly to beams, including weight or projecting beam stem above or below the slab.

13.6.6 FACTORED MOMENTS IN MIDDLE STRIPS

13.6.6.1 That portion of negative and positive factored moments not resisted by column strips shall be proportionally assigned to corresponding half middle strips.

13.6.6.2 Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

13.6.6.3 A middle strip adjacent to and parallel with an edge supported by a wall shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior support.

13.6.7 MODIFICATION OF FACTORED MOMENTS

Negative and positive factored moments may be modified by 10 percent provided the total static moment for a panel in the directions considered is not less than that required by Eq. (13-3).

13.6.8 FACTORED SHEAR IN SLAB SYSTEMS WITH BEAMS

13.6.8.1 Beams with $(\alpha_1 l_2/l_1)$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas bounded by 45 deg lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides.

13.6.8.2 Beams with $(\alpha_1 l_2/l_1)$ less than 1.0 may be proportioned to resist shear obtained by linear interpolation, assuming beams carry no load at $\alpha = 0$.

13.6.8.3 In addition to shears calculated according to Articles 13.6.8.1 and 13.6.8.2, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

13.6.8.4 Slab shear strength may be computed on the assumption that load is distributed to supporting beams in accordance with Article 13.6.8.1 or 13.6.8.2. Resistance to total shear occurring on a panel shall be provided.

13.6.8.5 Shear strength shall satisfy requirements of Sub-section 11.

13.6.9 FACTORED MOMENTS IN COLUMNS AND WALLS

13.6.9.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

13.6.9.2 At an interior support, suporting elements above and below the slab shall resist the moment specified by Eq. (13-4) in direct proportion to their stiffness unless a general analysis is made.

$$M = 0.07[(w_d + 0.5w_l)\ell_2\ell_n^2 - w'_d\ell'_2(\ell'_n)^2] \qquad (13\text{-}4)$$

where w'_d, l'_2 and l'_n refer to shorter span.

13.6.10 PROVISION FOR EFFECTS OF PATTERN LOADINGS

Where ratio $_a$ of dead load to live load is less than 2, one of the following conditions must be satisfied:

(a) The sum of flexural stiffnessess of the columns above and below the slab shall be such that is not less than $_{min}$ specified in Table 13.6.10.

(b) If $_c$ for the columns above and below the slab is less than $_{min}$, specified in Table 13.6.10, positive factored moments in panels supported by such columns shall be multiplied by the coefficient determined from Eq. (13-5).

TABLE 13.6.10
VALUES OF min

a	Aspect ratio l_2/l_1	Relative beam stiffness, 0	0.5	1.0	2.0	4.0
2.0	0.5 - 2.0	0	0	0	0	0
1.0	0.5	0.6	0	0	0	0
	0.8	0.7	0	0	0	0
	1.0	0.7	0.1	0	0	0
	1.25	0.8	0.4	0	0	0
	2.0	1.2	0.5	0.2	0	0
0.5	0.5	1.3	0.3	0	0	0
	0.8	1.5	0.5	0.2	0	0
	1.0	1.6	0.6	0.2	0	0
	1.25	1.9	1.0	0.5	0	0
	2.0	4.9	1.6	0.8	0.3	0
0.33	0.5	1.8	0.5	0.1	0	0
	0.8	2.0	0.9	0.3	0	0
	1.0	2.3	0.9	0.4	0	0
	1.25	2.8	1.5	0.8	0.2	0
	2.0	13.0	2.6	1.2	0.5	0.3

$$\delta_s = 1 + \frac{2 - \beta_a}{4 + \beta_a}\left(1 - \frac{\alpha_c}{\alpha_{min}}\right) \qquad (13\text{-}5)$$

where β_a is ratio of dead load to live load, per unit area (in each case without load factors).

13.7 **Equivalent frame method**

13.7.1 Design of slab systems by the Equivalent Frame Method shall be based on assumptions given in Articles 13.7.2 through 13.7.6, and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

13.7.1.1 Where metal column capitls are used, account may be taken of their contributions to stiffness and resistnce to moment and to shear.

13.7.1.2 Change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected.

13.7.2 EQUIVALENT FRAME

13.7.2.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building.

13.7.2.2 Each frame shall consist of a row of columns and supports and slab-beam strips, bounded laterally by the centerline of panel on each side of the centerline of columns or supports.

13.7.2.3 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members (Article 13.7.5) transverse to the direction of the span for which moments are being determined and extending to bounding lateral panel centerlines on each side of a column.

13.7.2.4 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of adjacent panel.

13.7.2.5 Each equivalent frame may be analyzed in its entirety, or for gravity loading, each floor and the roof (slab-beams) may be analyzed separately with far ends of columns considered fixed.

13.7.2.6 Where slab-beams are analyzed separately, it may be assumed in determining moment at a given support that the slab-beam is fixed at any support two panels distant therefrom, provided the slab continues beyond that point.

13.7.3 SLAB-BEAMS

13.7.3.1 Moment of inertia of slab-beams at any cross-section outside of joints or column capitals may be based on the gross area of concrete.

13.7.3.2 Variation in moment of inertia along axis of slab-beams shall be taken into account.

13.7.3.3 Moment of inertia of slab-beams from centre of column to face of column, bracket or capital shall be assumed equal to the moment of inertia of the slab-beam at face of column, bracket or capital divided by the quantity $(1 - c_2/l_2)^2$ where c_2 and l_2 are measured transverse to the direction of the span for which moments are being determined.

13.7.4 COLUMNS

13.7.4.1 Moment of intertia of columns at any cross-section outside of joints or column capitals may be based on the gross area of concrete.

13.7.4.2 Variation in moment of inertia along axis of columns shall be taken into account.

13.7.4.3 Moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed infinite.

13.7.5 TORSIONAL MEMBERS

13.7.5.1 Torsional members (Article 13.7.2.3) shall be assumed to have constant cross-section throughout their length consisting of the larger of

(a) A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined.

(b) For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab.

(c) Transverse beam as defined in Article 13.2.4.

13.7.5.2 Stiffness K_t of the torsional members shall be calculated by the following expression:

$$K_t = \sum \frac{9E_{cs}C}{\ell_2\left(1 - \frac{c_2}{\ell_2}\right)^3} \quad (13\text{-}6)$$

where c_2 and l_2 relate to the transverse spans on each side of column.

13.7.5.3 The constant C in Eq. (13-6) may be evaluated for the cross-section by dividing it into separate rectangular parts and carrying out the following summation:

$$C = \sum \left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3} \quad (13\text{-}7)$$

13.7.5.4 Where beams frame into columns in the direction of the span for which moments are being determined, value of K_t as computed by Eq. (13-6) shall be multiplied by the ratio of moment of inertia of slab with such beam to moment of inertia of slab without such beam.

13.7.6 ARRANGEMENT OF LIVE LOAD

13.7.6.1 When loading pattern is known, the equivalent frame shall be analyzed for that load.

13.7.6.2 When live load is variable but does not exceed three-quarters of the dead load, or the nature of live load is such that all panels will be loaded simultaneously, maximum factored moments may be assumed to occur at all sections with full factored live load on entire slab system.

13.7.6.3 For loading conditions other than those defined in Articles 13.7.6.2, maximum positive factored moment near midspan of a panel may be assumed to occur with three-quarters of the full factored live load on the panel and on alternate panels; and maximum negative factored moment in the slab at a support may be assumed to occur with three-quarters of the full live load on adjacent panels only.

13.7.6.4 Factored moments shall be taken not less than those occurring with full factored live load on all panels.

13.7.7 FACTORED MOMENTS

13.7.7.1 At interior supports, critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not greater than 0.175 l_1 from centre of a column.

13.7.7.2 At exterior supports provided with brackets or capitals, critical section for negative factored moment in the span perpendicular to an edge shall be taken at a distance from face of supporting element not greater than one-half the projection of bracket or capital beyond face of supporting element.

13.7.7.3 Circular or regular polygon shaped supports shall be treated as square supports with the same area for location of critical section for negative design moment.

13.7.7.4 Slab systems within limitations of Article 13.6.1 when analyzed by the Equivalent Frame Method, may have resulting computed moments reduced in such proportion that the absolute sum of the positive and average negative moments used in design need not exceed the value obtained from Eq. (13-3).

13.7.7.5 Moments at critical sections across the slab-beam strip of each frame may be distributed to column strips, beams and middle strips as provided in Articles 13.6.4, 13.6.5, and 13.6.6 if the requirement of Article 13.6.1.6 is satisfied.

SUB-SECTION 14 - WALLS

14.1 Scope

14.1.1 Provisions of Chapter 14 shall apply for design of walls subjected to axial load with or without flexure.

14.1.2 Cantilever retaining walls are designed according to flexural design provisions of Chapter 10 with minimum horizontal reinforcement according to Article 14.3.3.

14.2 General

14.2.1 Walls shall be designed for eccentric loads and any tensile lateral or other loads to which they are subjected.

14.2.2 Walls subject to axial loads shall be designed in accordance with Articles 14.2, 14.3 and either 14.4 or 14.5.

14.2.3 Design for shear shall be in accordance with Article 11.10.

14.2.4 Unless demonstrated by a detailed analysis, horizontal length of wall to be considered as effective for each concentrated load shall not exceed centre-to-centre distance between loads, nor width of bearing plus four times the wall thickness.

14.2.5 Compression members built integrally with walls shall conform to Article 10.8.2.

14.2.6 Walls shall be anchored to intersecting elements such as floors, roofs, or to columns, pilasters, buttresses and intersecting walls, and footings.

14.2.7 Quantity of reinforcement and limits of thickness required by Articles 14.3 and 14.5 may be waived where structural analysis shows adequate strength and stability.

14.2.8 Transfer of force to footing at base of wall shall be in accordance with Article 15.8.

14.3 Minimum reinforcement

14.3.1 Minimum vertical and horizontal reinforcement shall be in accordance with Article 14.3.2 and 14.3.3 unless a greater amount is required for shear by Articles 11.10.8 and 11.10.9.

14.3.2 Minimum ratio of vertical reinforcement area to gross concrete area shall be:

(a) 0.0012 for deformed bars not larger than No. 15 with a specified yield strength not less than 400 MPa, or
(b) 0.0015 for other deformed bars, or
(c) 0.0012 for welded wire fabric (smooth or deformed) not larger than W31 or D31.

14.3.3 Minimum ratio of horizontal reinforcement area to gross concrete area shall be:

(a) 0.0020 for deformed bars not larger than No. 15 with a specified yield strength not less than 400 MPa, or
(b) 0.0025 for other deformed bars, or
(c) 0.0020 for welded wire fabric (smooth or deformed) not larger than W31 or D31.

14.3.4 Walls more than 250 mm thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

(a) One layer consisting of not less than one-half and not more than two-thirds of total reinfocement required for each direction shall be placed not less than 50 mm or more than one-third the thickness of wall from exterior surface.

(b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 20 mm nor more than one-third the thickness of wall from interior suface.

14.3.5 Vertical and horizontal reinforcement shall not be spaced further apart than three times the wall thickness, nor 500 mm.

14.3.6 Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

14.3.7 In addition to the minimum reinforcement required by Articles 14.3.1 and 14.3.2, not less than two No. 15 bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the corners of the openings but not less than 600 mm.

14.4 Walls designed as compression members

Except as provided in Article 14.5, walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with provisions of Articles 10.2, 10.3, 10.10, 10.11, 10.12, 10.15, and Articles 14.2 and 14.3.

14.5 Empirical design method

14.5.1 Walls of solid rectangular cross-section may be designed by the empirical provisions of Article 14.5 if resultant of all factored loads is located within the middle-third of the overall thickness of wall and all limits of Articles 14.2, 14.3, and 14.5 are satisfied.

14.5.2 Design axial load strength of a wall satisfying limitations of Article 14.5.1 shall be computed by Eq. (14-1) unless designated in accordance with Article 14.4.

$$\phi P_{nw} = 0.55\,\phi\,f'_c\,A_g\left[1-\left(\frac{k\ell_c}{32h}\right)^2\right] \qquad (14\text{-}1)$$

where ϕ = 0.70 and effective length factor k shall be:

For walls braced top and bottom against lateral translation and

(a) restrained against rotation at one or both ends (top and/or bottom) 0.8

(b) unrestrained against rotation at both ends 1.0

For walls not braced against lateral translation 2.0

14.5.3 MINIMUM THICKNESS OF WALLS DESIGNED BY EMPIRICAL DESIGN METHOD

14.5.3.1 Thickness of bearing walls shall not be less than 1/25 the supported length of height, whichever is shorter, nor less than 100 mm.

14.5.3.2 Thickness of exterior basement walls and foundation walls shall not be less than 190 mm.

14.6 **Nonbearing walls**

14.6.1 Thickness of nonbearing walls shall not be less than 100 mm, nor less than 1/30 the least distance between members that provide lateral support.

14.7 **Walls as grade beams**

14.7.1 Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of Articles 10.2 through 10.7. Design for shear shall be in accordance with provisions of Chapter 11.

14.7.2 Portions of grade beam walls exposed above grade shall also meet requirements of Article 14.3.

SUB-SECTION 15 - FOOTINGS

15.1 **Scope**

15.1.1 Provisions of Chapter 15 shall apply for design of isolated footings and where applicable, to combined footings and mats.

15.1.2 Additional requirements for design of combined footings and mats are given in Article 15.10.

15.2 **Loads and reactions**

15.2.1 Footings shall be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of this Code and as provided in Chapter 15.

15.2.2 Base area of footing or number and arrangement of piles shall be determined from unfactored forces and moments transmitted by footing to soil or piles and permissible soil pressure or permissible pile capacity selected through principles of soil mechanics.

15.2.3 For footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at pile center.

15.3 **Footings supporting circular or regular polygon shaped columns or pedestals**

Circular or regular polygon shaped concrete columns or pedestals may be treated as square members with the same area for location of critical sections for moment, shear, and development of reinforcement in footings.

15.4 Moment in footings

15.4.1 External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over entire area of footing on one side of that vertical plane.

15.4.2 Maximum factored moment for an isolated footing shall be computed as prescribed in Article 15.4.1 at critical sections located as follows:

(a) At face of column, pedestal or wall, for footings supporting a concrete column, pedestal or wall,
(b) Halfway between middle and edge of wall, for footings supporting a masonry wall.
(c) Halfway between face of column and edge of steel base plate, for footings supported a column with steel base plate.

15.4.3 In one-way footings, and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

15.4.4 In two-way rectangular footings, reinforcement shall be distributed as follows:

15.4.4.1 Reinforcement in long direction shall be distributed uniformly across entire width of footing.

15.4.4.2 For reinforcement in short direction, a portion of the total reinforcement given by Eq. (15-1) shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction shall be distributed uniformly outside centre band width of footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)} \qquad (15\text{-}1)$$

15.5 Shear in footings

15.5.1 Shear strength of footings shall be in accordance with Article 11.11.

15.5.2 Location of critical section for shear in accordance with Chapter 11 shall be measured from face of column, pedestal, or wall, for footings supporting a column or pedestal with steel base plates, the critical section shall be measured from location defined in Article 15.4.2.(c).

15.5.3 Computation of shear on any section through a footing supported on piles shall be in accordance with the following:

15.5.3.1 Entire reaction from any pile whose centre is located $d_p/2$ or more outside the section shall be considered as producing shear on that section.

15.5.3.2 Reaction from any pile whose centre is located $d_p/2$ or more inside the section shall be considered as producing no shear on that section.

15.5.3.3 For intermediate positions of pile centre, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

15.6 Development of reinforcement in footings

15.6.1 Development of reinforcement in footings shall be in accordance with Chapter 12.

15.6.2 Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by embedment length, hook (tension only) or mechanical device, or a combination thereof.

15.6.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in Article 15.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. See also Article 12.10.6.

15.7 Minimum footing depth

Depth of footing above bottom reinforcement shall not be less than 150 mm for footings on soil, nor less than 300 mm for footings on piles.

15.8 Transfer of force at base of column, wall or reinforced pedestal

15.8.1 Forces and moments at base of column, wall or pedestal shall be transferred to supporting pedestal or footing by bearing on concrete and by reinforcement, dowels, and mechanical connectors.

15.8.1.1 Bearing on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by Article 10.15.

15.8.1.2 Reinforcement, dowels or mechanical connectors between supported and suporting members shall be adequate to transfer:

(a) All compressive force that exceeds concrete bearing strength of either member,

(b) Any computed tensile force across interface.

In addition, reinforcement, dowels, or mechanical connectors shall satisfy Article 15.8.2 or 15.8.3.

15.8.1.3 If calculated moments are transferred to supporting pedestal or footing, reinforcement, dowels or mechanical connectors shall be adequate to satisfy Article 12.17.

15.8.1.4 Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of Article 11.7 or by other appropriate means.

15.8.2 In cast-in-place construction, reinforcement required to satisfy Article 15.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

15.8.2.1 For cast-in-place columns and pedestals, area of reinforcement across interface shall be not less than 0.005 times gross area of supported member.

15.8.2.2 For cast-in-place walls, area of reinforcement across interface shall not be less than minimum vertical reinforcement given in Article 14.3.2.

15.8.2.3 Diameter of dowels, if used, shall not exceed diameter of longitudinal bars by more than 5 mm.

15.8.2.4 At footings, No.45 and No.55 longitudinal bars in compression only, may be lap spliced with dowels to provide reinforcement required to satisfy Article 15.8.1. Dowels shall not be larger than No.35 bar and shall extend into supported member a distance not less than the development length of No.45 or No.55 bars, or the splice length of the dowels, whichever is greater, and into the footing a distance not less than the development length of the dowels.

15.8.2.5 If a pinned or rocker connection is provided in cast-in-place construction, connection shall conform to Article 15.8.1 and 15.8.3.

15.8.3 In precast construction, reinforcement required to satisfy Article 15.8.1 may be provided by anchor bolts or suitable mechanical connectors.

15.8.3.1 Connection between precast columns or pedestals and supporting member shall have a tensile strength not less than 1.5 A_g in Newtons, where A_g is cross-sectional area of wall.

15.8.3.2 Connection between precast wall and supporting member shall have a tensile strength not less than $A_g/3$ in Newtons, where A_g is cross-sectional area of wall.

15.8.3.3 Anchor bolts and mechanical connectors shall be designed to reach their design strength prior to anchorage failure or failure of surrounding concrete.

15.9 Sloped or stepped footings

15.9.1 In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section. (See also Article 12.10.6)

15.9.2 Sloped or stepped footings designed as a unit shall be constructed to assure action as a unit.

15.10 Combined footings and mats

15.10.1 Footings supporting more than one column, pedestal or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of this Code.

15.10.2 The Direct Design Method of Chapter 13 shall not be used for design of combined footings and mats.

15.10.3 Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

SUB-SECTION 16 - PRE-CAST CONCRETE

16.1 Scope

16.1.1 Provision of Chapter 16 shall apply for design of pre-cast concrete members defined as concrete elements cast elsewhere than their final position in the structure.

16.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Chapter 16 shall apply to pre-cast concrete.

16.2 Design

16.2.1 Design of pre-cast members shall consider all loading and restraint conditions from initial fabrication to completion of the structure, including form removal, storage, transportation, and erection.

16.2.2 In pre-cast construction that does not behave monolithically, effects at all interconnected and adjoining details shall be considered to assure proper performance of the structural system.

16.2.3 Effects of initial and long-time deflections shall be considered, including effects on interconnected elements.

16.2.4 Design of joints and bearings shall include effects of all forces to be transmitted, including shrinkage, creep, temperature, elastic deformation, wind and earthquake.

16.2.5 All details shall be designed to provide for manufacturing and erection tolerances and temporary erection stressed.

16.3 Pre-cast wall panels

16.3.1 Pre-cast bearing and nonbearing walls shall be designed in accordance with provisions of Chapter 14.

16.3.2 Where pre-cast panels are designed to span horizontally to columns or isolated footings, the ratio of height to thickness shall not be limited, provided the effect of deep beam action, lateral buckling, and deflections are provided for in the design. See Article 10.7.

16.4 Details

16.4.1 All details of reinforcement connections, bearing seats, inserts, anchors, concrete cover, openings, lifting devices, fabrication and erection tolerances shall be shown on the shop drawings.

16.4.2 When approved by the Engineer, embedded items (such as dowels, or inserts) that either protrude from concrete or remain exposed for inspection may be embedded while concrete is in a plastic state provided.

16.4.2.1 Embedded items shall not be required to be hooked or tied to reinforcement within plastic concrete.

16.4.2.2 Embedded items shall be maintained in correct position while concrete remains plastic.

16.4.2.3 Embedded items shall be properly anchored to develop required factored loads.

6.5 Identification and marking

16.5.1 Each pre-cast member or element shall be marked to indicate location in the structure, top surface, and date of fabrication.

16.5.2 Identification marks shall correspond to the placing plans.

16.6 Transportation, storage, and erection

16.6.1 During curing, for removal, storage, transportation, and erection, pre-cast members shall not be over-stressed, warped or otherwise damaged or have camber adversely affected.

16.2.2 Pre-cast members shall be adequately braced and supported during erection to insure proper alignment and structural integrity until permanent connections are completed.

SUB-SECTION 17 - COMPOSITE CONCRETE FLEXURAL MEMBERS

17.1 Scope

17.1.1 Provisions of Chapter 17 shall apply for design of composite concrete flexural members defined as pre-cast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to loads as a unit.

17.1.2 All provisions of this Code shall apply to composite concrete flexural members, except as specifically modified in Chapter 17.

17.2 General

17.2.1 An entire composite member or portions thereof may be used in resisting shear and moment.

17.2.2 Individual elements shall be investiated for all critical states of loading.

17.2.3 If the specified strength, unit mass, or other properties of the various elements are different, properties of the individual element or the most critical values, shall be used in design.

17.2.4 In strength computations of composite members, no distinction shall be made between shored and unshored members.

17.2.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

17.2.6 Reinforcement shall be provided as required to control cracking and to prevent separation of individual elements of composite members.

17.2.7 Composite members shall meet requirements for control of deflections in accordance with Article 9.5.5.

17.3 Shoring

When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and cracking at time of shoring removal.

17.4 Vertical shear strength

17.4.1 When an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Chapter 11 as for a monolithically cast member of the same cross-sectional shape.

17.4.2 Shear reinforcement shall be fully anchored into interconnected elements in accordance with Article 12.13.

17.4.3 Extended and anchored shear reinforcement may be included as ties for horizontal shear.

17.5 Horizontal shear strength

17.5.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

17.5.2 Unless calculated in accordance with Article 17.5.3, design of cross-sections subject to horizontal shear shall be based on

$$V_u \leq \phi V_{nh} \qquad (17\text{-}1)$$

Where V_u is factored shear force at section considered and V_{nh} is nominal horizontal shear strength in accordance with the following:

17.5.2.1 When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $0.6b_vd$ in Newtons.

17.5.2.2 When minimum ties are provided in accordance with Article 17.6, and contact surfaces are clean, free of laitance but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $0.6b_vd$ in newtons.

17.5.2.3 When minimum ties are provided in accordance with Article 17.6, and contact surfaces are clean, free of laitance, not intentionally roughened to a full amplitude of approximately 5 mm, shear strength V_{nh} shall not be taken greater than $2.5b_vd$ in newtons.

17.5.2.4 When factored shear force V_u at section considered exceeds $\emptyset(2.5b_vd)$ design for horizontal shear shall be in accordance with Article 11.7.

17.5.3 Horizontal shear may be investigated by computing the actual change in compressive or tensile force in an segment, and provisions made to transfer that force as horiontal shear to the supporting element. The factored horizontal shear force shall not exceed horizontal shear strength V_{nh} as given in Articles 17.5.2.1 through 17.5.2.4 where area of contact surface A_c shall be substituted for b_vd.

17.5.4 When tension exists across any contact surface between interconnected elements, shear transfer by contact may be assumed only when minimum ties are provided in accordance with Article 17.6.

17.6 Ties for horizontal shear

17.6.1 When ties are provided to transfer horizontal shear, the area shall not be less than that required by Article 11.5.5.3 and tie spacing shall not exceed four times the least dimension of supported element, nor 600 mm.

17.6.2 Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs or welded wire fabric (smooth or deformed).

17.6.3 All ties shall be fully anchored into interconnected elements in accordance with Article 12.13.

SUB-SECTION 18 - PRE-STRESSED CONCRETE

18.1 Scope

18.1.1 Provisions of Sub-Section 18 shall apply to members pre-stressed with wire, strands, or bars conforming to provisions for pre-stressing tendons in Article 3.5.5.

18.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Chapter 18, shall apply to pre-stressed concrete.

18.1.3 The following provisions of this Code shall not apply to pre-stressed concrete, except as specifically noted; Articles 8.4, 8.10.2, 8.10.3, 8.10.4, 8.11, 10.3.2, 10.3.3, 10.5, 10.6, 10.9.1, 10.9.2, Sub-Section 13 and Articles 14.3, 14.5, and 14.6.

18.2 General

18.2.1 Pre-stressed members shall meet the strength requirements specified in this Code.

18.2.2 Design of pre-stressed members shall be based on strength and on behaviour at service conditions at all load stages that may be critical during the life of the structure from the time pre-stress is first applied.

18.2.3 Stress concentrations due to pre-stressing shall be considered in design.

18.2.4 Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length and rotations due to pre-stressing. Effects of termperature and shrinkage shall also be included.

18.2.5 Possibility of buckling in a member between points where concrete and pre-stressing tendons are in contact and of buckling in thin webs and flanges shall be considered.

18.2.6 In computing section properties prior to bonding of pre-stressing tendons, effect of loss of area due to open ducts shall be considered.

18.3 Design assumptions

18.3.1 Strength design of pre-stressed members for flexure and axial loads shall be based on assumptions given in

Article 10.2, except Article 10.2.4 shall apply only to reinforcement conforming to Article 3.5.3.

18.3.2 For investigation of stresses at transfer of pre-stress, at service loads, and at cracking loads, straight-line theory may be used with the following assumptions:

18.3.2.1 Strains vary linearly with depth through entire load range.

18.3.2.2 At cracked sections, concrete resists no tension.

18.4 Permissible stresses in concrete - Flexural members

18.4.1 Stresses in concrete immediately after pre-stress transfer (before time-dependent pre-stress losses) shall not exceed the following:

(a) Extreme fibre stress in compression ....$0.60\ f'_{ci}$

(b) Extreme fibre stress in tension except as permitted in (c) ....$0.25\sqrt{f'_{ci}}$

(c) Extreme fibre stress in tension at ends of simply supported members ....$0.5\ \sqrt{f'_{ci}}$

Where computed tensile stresses exceed these values, bonded auxiliary reinforcement (non-pre-stressed or pre-stressed) shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

18.4.2 Stresses in concrete at service loads (after allowance for all pre-stress losses) shall not exceed the following:

(a) Extreme fibre stress in compression ....$0.45\ f'_c$

(b) Extreme fibre stress in tension in precompressed tensile zone ....$0.5\sqrt{f'_c}$

(c) Extreme fibre stress in tension in pre-compressed tensile zone of members (except two-way slab systems) where analysis based on transformed cracked sections and on bilinear moment-deflection relationships show that immediate and long-time deflections comply with requirements of Article 9.5.4, and where cover requirements comply with Article 7.7.3.2. .... $\sqrt{f'_c}$

18.4.3 Permissible stresses in concrete of Article 18.4.1 and 18.4.2 may be exceeded is shown by test or analysis that performance will not be impaired.

18.5 Permissible stresses in pre-stressing tendons

18.5.1 Tensile stress in pre-stressing tendons shall not exceed the following:

(a) Due to tendon jacking force ....$0.94f_{py}$

but not greater than $0.85f_{pu}$ or maximum value recommended by manufacturer of pre-stressing tendons or anchorages.

(b) Immediately after pre-stress transfer ......$0.82f_{py}$

but not greater than $0.74f_{pu}$.

(c) Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage ....$0.70f_{pu}$

18.6 Loss of pre-stress

18.6.1 To determine effective pre-stress f_{se}, allowance for the folowing sources of loss of pre-stress shall be considered:

(a) Anchorage seating loss
(b) Elastic shortening of concrete
(c) Creep of concrete
(d) Shrinkage of concrete
(e) Relaxation of tendon stress
(f) Friction loss due to intended or unintended curvature in post-tensioning tendons.

18.6.2 FRICTION LOSS IN POST-TENSIONING TENDONS

18.6.2.1 Effect of friction loss in post-tensioning tendons shall be computed by

$$P_s = P_x e^{(K\ell_x + \mu\alpha)} \qquad (18\text{-}1)$$

When (Kl_x +) is not greater than 0.3, effect of friction loss may be computed by

$$P_s = P_x(1 + K\ell_x + \mu\alpha) \qquad (18\text{-}2)$$

18.6.2.2 Friction loss shall be based on experimentally determined wobble K and curvature friction coefficients, and shall be verified during tendon stressing operations.

18.6.2.3 Values of wobble and curvature friction coefficients used in design shall be shown on design drawings.

18.6.3 Where loss of pre-stress in a member may occur due to connection of member to adjoining construction, such loss of pre-stress shall be allowed for in design.

18.7 **Flexural stength**

18.7.1 Design moment strength of flexural members shall be computed by the strength design methods of this Code. For pre-stressing tendons, f_{ps} shall be substituted for f_y in strength computations.

18.7.2 In lieu of a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} shall be used if f_{se} is not less than $0.5f_{pu}$.

(a) For members with bonded pre-stressing tendons:

$$f_{ps} = f_{pu}\left(1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]\right) \qquad (18\text{-}3)$$

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. (18-3), the term

$$\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]$$

shall be taken not less than 0.17 and d' shall be no greater than $0.15d_p$.

(b) For members with unbonded pre-stressing tendons and with a span-to-depth ratio of 35 or less:

$$f_{ps} = f_{se} + 70 + \frac{f'_c}{100\rho_p} \qquad (18\text{-}4)$$

but f_{ps} in Eq (18-4) shall not e taken greater than f_{py} nor (f_{se} + 400).

(c) For members with unbonded prestressing tendons and with a span-to-ratio depth greater than 35:

$$f_{ps} = f_{se} + 70 + \frac{f'_c}{300\rho_p} \qquad (18\text{-}5)$$

but f_{ps} in Eq. (18-5) shall not be taken greater than f_{py}, nor (f_{se} + 200).

18.7.3 Non-pre-stressed reinforcement conforming to Article 3.5.3 if used with pre-stressing tendons may be considered to contribute to the tensile force and may be included in monent strength computations at a stress equal to the specified yield strength f_y. Other non-pre-stressed reinforcement may be included in strength computations only if a strain comptibility analysis is made to determine stresses in such reinforcement.

18.8 Limits for reinforcement of flexural members

18.8.1 Ratio of pre-stressed and non-pre-stressed reinforcement used for comutation of moment strength of a member, except as provided in Article 18.8.2, shall be such that ω_p, $[\omega_p + d/d_p(\omega - \omega')]$ or $[\omega_{pw} + d/d_p(\omega_w - \omega'_w)]$ is not greater than $0.36\beta_1$.

18.8.2 When a reinforcement ratio in excess of that specified in Article 18.8.1 is provided, design moment strength shall not exceed the moment strength based on the compression portion of the moment couple.

18.8.3 Total amount of pre-stressed and non-pre-stressed reinforcement shall be adequate to develop factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture f_r, specified in Article 9.5.2.3. except for flexural members with shear and flexural strength at least twice that required by Article 9.2.

18.9 Minimum bonded reinforcement

18.9.1 A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded pre-stressing tendons as required by Articles 18.9.2 and 18.9.3.

18.9.2 Except as provided in Article 18.9.3 minimum area of bonded reinforcement shall be computed by:

$$A_s = 0.004A \qquad (18\text{-}6)$$

18.9.2.1 Bonded reinforcement required by Eq. (18-6) shall be uniformly distributed over pre-compressed tensile zone as close as practicable to extreme tension fibre.

18.9.2.2 Bonded reinforcement shall be required regardless of service load stress conditions.

18.9.3 For two-way flat plates, defined as solid slabs of uniform thickness, minimum area and distribution of bonded reinforcement shall be as follows:

18.9.3.1 Bonded reinforcement shal not be required in positive moment areas where computed tensile stress in concrete at service load (after allowance for all pre-stress losses) does not exceed $\sqrt{f'_c}/6$.

18.9.3.2 In positive moment areas where computed tensile stress in concrete at service load exceeds $f'_c/6$, minimum area of bonded reinforcement shall be computed by:

$$A_s = \frac{N_c}{0.5f_y} \qquad (18\text{-}7)$$

where design yield strength f_y shall not exceed 400 MPa. Bonded reinforcement shall be uniformly distributed over pre-compressed tensile zone as close as practicable to extreme tensile fibre.

18.9.3.3 In negative moment areas at column supports, minimum area of bonded reinforcement in each direction shall be computed by:

$$A_s = 0.00075h\ell \qquad (18\text{-}8)$$

where l is length of span in direction parallel to that of the reinforcement being determined. Bonded reinforcement required by Eq. (18-8) shall be distributed within a slab width between lines that are 1.5h outside opposite faces of the column support. At least four bars or wires shall be provided in each direction. Spacing of bonded reinforcement shall not exceed 300 mm.

18.9.4 Minimum length of bonded reinforcement required by Articles 18.9.2 and 18.9.3 shall be as follows.

18.9.4.1 In positive moment areas, minimum length of bonded reinforcement shall be one-third the clear span length and centered in positive moment area.

18.9.4.2 In negative moment areas, bonded reinforcement shall extend one-sixth the clear span on each side of support.

18.9.4.3 Where bonded reinforcement is provided for design moment strength in accordance with Article 18.7.3, or for tensile stress conditions in accordance with Article 18.9.3.2, minimum length also shall conform to provisions of Sub-section 12.

18.10 Statically Indeterminate Structures

18.10.1 Frames and continuous construction of pre-stressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

18.10.2 Performance at service load conditions shall be determined by elastic analysis, considering reactions, moments, shears and axial forces produced by pre-stressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

18.10.3 Moments to be used to compute required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads including redistribution as permitted in Article 18.10.4.

18.10.4 REDISTRIBUTION OF NEGATIVE MOMENTS DUE TO GRAVITY LOADS IN CONTINUOUS PRE-STRESSED FLEXURAL MEMBERS

18.10.4.1 Where bonded reinforcement is provided at supports in accordance with Article 18.9.2, negative moments calculated by elastic theory for any assumed loading arrangement may be increased or decreased by not more than

$$20\left[1 - \frac{\omega_p + \frac{d}{d_p}(\omega - \omega')}{0.36\beta_1}\right] \text{percent}$$

18.10.4.2 The modified negative moments shall be used for calculating moments at sections within spans for the same loading arrangement.

18.10.4.3 Redistibution of negative moments shall be made only when the section at which moment is reduced is so designed that ω_p, $\left[\omega_p + d/d_p\ (\omega - \omega')\right]$, or $\left[\omega_{pw} + d/d_p\ (\omega_w - \omega_w')\right]$ whichever is applicable, is not greater than 0.24_{1}.

18.11 Compression members - Combined flexure and axial loads

18.11.1 Pre-stressed concrete members subject to combined flexure and axial load, with or without non-pre-stressed reinforcement, shall be proportioned by the strength design methods of this Code for members without pre-stressing. Effects or pre-stress, creep, shinkage, and temperature change shall be included.

18.11.2 LIMITS FOR REINFORCEMENT OF PRE-STRESSED COMPRESSION MEMBERS

18.11.2.1 Members with average pre-stress f_{pc} less than 1.5 MPa shall have a minimum reinforcement in accordance with Articles 7.10, 10.9.1, and 10.9.2 for columns of Article 14.3 for walls.

18.11.2.2 Except for walls, members with average pre-stress, f_{pc} equal to or greater than 1.5 MPa shall have all pre-stressing tendons enclosed by spirals or lateral ties in accordance with the following:

(a) Spirals shall conform to Article 7.10.4.
(b) Lateral ties shall be at least No. 10 in size or welded wire fabric of equivalent area, and spaced vertically not to exceed 48 tie bar or wire diameters, or least dimension of compression member.

(c) Ties shall be located vertically not more than half a tie spacing above top of footing or slab in any story, and shall be spaced as provided herein to not more than half a tie spacing below lowest horizontal reinforcement in members, supported above.
(d) Where beams or brackets frame into all sides of a column, ties may be terminated not more than 80 mm below lowest reinforcement in such beams or brackets.

18.11.2.3 For walls with average pre-stress f_{pc} equal to or greater than 1.5 MPa, a minimum reinforcement required by Article 14.3 may be waived where structural analysis shows adequate strength and stability.

18.12 Slab systems

18.12.1 Factored moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with provisions of Article 13.7 (excluding Articles 13.7.7.4 and 13.7.7.5) or by more detailed design procedures.

18.12.2 Moment strength of pre-stressed slabs at every section shall be at least equal to the required strength considering Articles 9.2, 9.3, 18.10.3 and 18.10.4. Shear strength of pre-stressed slabs at columns shall be at least equal to the required strength considering Articles 9.2, 9.3, 11.1, 11.11.2, and 11.12.2.4.

18.12.3 At service load conditions, all serviceability limitations, including specified limits on deflections, shall be met, with appropriate consideration of the factors listed in Article 18.10.2.

18.12.4 For normal live loads and loads uniformly distributed, spacing of pre-stressing tendons or groups of tendons in one direction shall not exceed 8 times the slab thickness, nor 1.5 m. Spacing of tendons shall also provide a minimum average pre-stress (after allowance for all pre-stress losses) of 0.9 MPa on the slab section tributary to the tendon to tendon group. A minimum of two tendons shall be provided in each direction through the critical shear section over columns. Special consideration of tendon spacing shall be provided for slabs with concentrated loads.

18.12.5 In slabs with unbonded pre-stressing tendons, bonded reinforcement shall be provided in accordance with Articles 18.9.3 and 18.9.4.

18.13 Tendon anchorage zones

18.13.1 Reinforcement shall be provided where required in tendon anchorage zones to resist bursting, splitting, and spalling forces induced by tendon anchorages. Regions of abrupt change in section shall be adequately reinforced.

18.13.2 End blocks shall be provided where required for support bearing or for distribution of concentrated pre-stressing forces.

18.13.3 Post-tensioning anchorages and supporting concrete shall be designed to resist maximum jacking force for strength of concrete at time of pre-stressing.

18.13.4 Post-tensioning anchorage zones shall be designed to develop the guaranteed ultimate tensile strength of pre-stressing tendons using a strength reduction factor ϕ of 0.90 for concrete.

18.14 Corrosion protection for unbonded pre-stressing tendons

18.14.1 Unbonded tendons shall be completely coated with suitable material to ensure corrosion protection.

18.14.2 Tendon wrapping shall be continuous over entire length to be unbonded, and shall prevent intrusion of cement paste or loss of coating materials during concrete placement.

18.15 Post-tensioning ducts

18.15.1 Ducts for grouted or unbonded tendons shall be mortar-tight and non-reactive with concrete, tendons, or filler material.

18.15.2 Ducts for grouted single wire, strand or bar tendons shall have an inside diameter at least 6 mm larger than tendon diameter.

18.15.3 Ducts for grouted multiple wire, strand, or bar tendons shall have an inside cross-sectional area at least two-times area of tendons.

18.15.4 Ducts shall be maintained free of water if members to be grouted are exposed to temperatures below freezing prior to grouting.

18.16 Grout for bonded pre-stressing tendons

18.16.1 Grout shall consist of Portland cement and water; or Portland cement, sand and water.

18.16.2 Materials for grout shall conform to the following:

18.16.2.1 Portland cement shall conform to Article 3.2.

18.16.2.2 Water shall conform to Article 3.4.

18.16.2.3 Sand, if used, shall conform to "Standard Specificaiton for Aggregate for Masonry Mortar" (ASTM C 144) except that graduation may be modified as necessary to obtain satisfactory workability.

18.16.2.4 Admixtures conforming to Article 3.6 and known to have no injurious effects on grout, steel, or concrete may be used. Calcium chloride shall not be used.

18.16.3 SELECTION OF GROUT PROPORTIONS

18.16.3.1 Proportions of materials for grout shall be based on either of the following:

(a) Results of tests on fresh and hardened grout prior to beginning grouting operations, or
(b) Prior documented experience with similar materials and equipment and under comparable field conditions.

18.16.3.2 Cement used in the work shall correspond to that on which selection of grout proportions was based.

18.16.3.3 Water content shall be minimum necessary for proper pumping of grout: however, water-cement ratio shall not exceed 0.45 by weight.

18.16.3.4 Water shall not be added to increase grout flowbility that has been decreased by delayed use of grout.

18.16.4 MIXING AND PUMPING GROUT

18.16.4.1 Grout shall be mixed in equipment capable of continuous mechanical mixing and agitation that will produce uniform distribution of materials, passed through screens, and pumped in a manner that will completely fill tendon ducts.

18.16.4.2 Temperature of members at time of grouting shall be above $2^{o}C$ and shall be maintained above $2^{o}C$ until field-cured 50 mm cubes of grout reach a minimum compressive strength of 6 MPa.

18.16.4.3 Grout temperatures shall not be above 30 C during mixing and pumping.

18.17 Protection of pre-stressing tendons

Burning or welding operations in vicinity of pre-stressing tendons shal be carefuly performed, so that tendons are not subject to excessive temperatures, welding sparks, or ground currents.

18.18 **Application and measurement of pre-stressing force**

18.18.1 Pre-stressing force shall be determined by both of the following methods:

(a) Measurement of tendon elongation, Required elongation shall be determined from average load-elongation on curves for pre-stressing tendons.
(b) Observation of jacking force on a calibrated gauge, or load call, or by use of a calibrated dynamometer.

Cause of any difference in fore determination between (a) and (b) that exceeds 5 percent shall be ascertained and corrected.

18.18.2 Where transfer of force from bulkheads of pre-tensioning bed to concrete is accomplished by flame cutting pre-stressing tendons, cutting points and cutting sequence shall be predetermined to avoid undesired temporary stresses.

18.18.3 Long lengths of exposed pre-tensioned strand shall be cut near the member of minimize shock to concrete.

18.18.4 Total loss of pre-stress due to unreplaced broken tendons shall not exceed 2 percent of total pre-stress.

18.19 **Post-tensioning anchorages and couplers**

18.19.1 Anchorages for unbonded pre-stressing tendons and couplers shall develop the specified ultimate strength of the tendons without exceeding anticipated set.

18.19.2 Anchorages for bonded pre-stressing tendons shal develop at least 90 percent of the specified ultimate strength of the tendons, when tested in an unbonded condition, without exceeding anticipated set. However, 100 percent of the specified ultimate strength of the tendons shall be developed after tendons are bonded in member.

18.19.3 Couplers shall be placed in areas approved by the Engineer and enclosed in housing long enough to permit necessary movements.

18.19.4 In bonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in anchorages in couplers.

18.19.5 Anchorage and end fittings shall be permanently protected against corrosion.

SUB-SECTION 19 - SHELLS AND FOLDED PLATE MEMBERS

19.1 Scope and definitions

19.1.1 Provisions of Sub-section 19 shape apply to thin-shell and folded plate concrete structures, including ribs and edge members.

19.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Chapter 19 shall apply to thin-shell structures.

19.1.3 THIN SHELLS - three dimensional spatial structures made up of one or more curved slabs or folded plates, whose thicknesses are small compared to their other dimensions. Thin shells are characterized by their three-dimensional load-carrying behaviour which is determined by the geometry of their forms, by the manner in which they are supported, and by the nature of the applied load.

19.1.4 FOLDED PLATES - a special class of shell structures formed by joining flat, thin slabs along their edges so as to create a three-dimensional spatial structure.

19.1.5 RIBBED SHELLS - Spatial structures with material placed primarily along certain preferred rib lines, with the area between the ribs filled with thin slabs or left open.

19.1.6 AUXILIARY MEMBERS - Ribs or edge beams which serve to strengthen, stiffen, and/or support the shell: usually, auxilliary members act jointly with the shell.

19.1.7 ELASTIC ANALYSIS - An analysis of deformations and internal forces based on equilibrium, compatibility of strains, and assumed elastic behaviour, and representing to suitable approximation the three-dimensional action of the shell together with its auxiliary members.

19.1.8 INELASTIC ANALYSIS - An analysis of deformations and internal forces based on equilibrium, nonlinear stress-strain relations for concrete and reinforcement, consideration of cracking and time dependent effects, and compatibility of strains. The analysis shall represent to suitable approximation the three-dimensional action of the shell together with its auxiliary members.

19.1.9 EXPERIMENTAL ANALYSIS - An analysis procedure based on the measurement of deformations and/or strains of the structure as its model, experimental analysis may be based on either elastic or inelastic behaviour.

19.2 Analysis and design

19.2.1 Elastic behaviour shall be an accepted basis for determining internal forces, and displacements, of thin shells. This behaviour may be established by computations based on an analysis of the uncracked concrete structure in which the material is assumed linearly elastic, homogenous and isotropic. Poisson's ratio of concrete may be assumed equal to zero.

19.2.2 Inelastic analysis may be used where it can be shown that such methods provide a safe basis for design.

19.2.3 Equilibrium checks of internal resistances and external loads shall be made to ensure consistency of results.

19.2.4 Experimental or numerical analysis procedures may be used where it can be shown that such procedures provide a safe basis for design.

19.2.5 Approximate methods of analysis not satisfying compatibility of strains either within the shell or between the shell and auxiliary members may be used where it can be shown that such methods provide a safe basis for design.

19.2.6 In pre-stressed shells, the analysis must also consider behaviour under loads induced during pre-stressing, at cracking load, and at factored load. Where pre-stressing tendons are draped within a shell, design shall take into account force components on the shell resulting from tendon profile not lying in one plane.

19.2.7 The thickness h of a thin shell, and its reinforcement, shall be proportioned for the required strength and serviceability. All elements shall be proportioned by the same method, using either the strength design method of Article 8.1.1 or the alternate design method of Article 8.1.2.

19.2.8 Shell design shall investigate and preclude the possibility of general or local instability.

19.2.9 Auxiliary members shall be designed according to the applicable provisions of this Code. The design method selected for shell elements under Article 19.2.7 shall also be used for auxiliary members. A portion of the shell equal to the flange width specified in Article 8.10 may be assumed to act with the auxiliary member. In such portions of the shell, the reinforcement perpendicular to the auxiliary member shall be at least equal to that required for the flange of a T-beam by Article 8.10.5.

19.3 Design strength of materials

19.3.1 Specified compressive strength of concrete f'_c at 28 days shall not be less than 20 MPa.

19.3.2 Specified yield strength of non-pre-stressed reinforcement f_y shall not exceed 400 MPa.

19.4 Shell reinforcement

19.4.1 Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces, to resist bending and twisting moments, to control shrinkage and temperature cracking, and as special reinforcement at shell boundaries, load attachments, and shell openings.

19.4.2 Membrane reinforcement shall be provided in two or more directions in all parts of the shell.

19.4.3 The area of shell reinforcement at any section as measured in two orthogonal directions shall not be less than the slab shrinkage or temperature reinforcement required by Article 7.12.

19.4.4 Reinforcement required to resist shell membrane forces shall be provided so that the design strength in every direction shall be at least equal to the component of the principal membrane forces in the same direction due to factored loads.

19.4.5 The area of shell tension reinforcement shall be limited so that the reinforcement will yield before crushing of concrete in compression can take place.

19.4.6 In regions of high tension, membrane reinforcement shall, if practical, be placed in the general directions of the principal tensile membrane forces. Where this is not practical, membrane reinforcement may be placed in two or more component directions.

19.4.7 If the direction of reinforcement varies more than 10 deg from the direction of principal tensile membrane force, the amount of reinforcement may have to be increased to limit the width of possible cracks at service load levels.

19.4.8 Where the magnitude of the principal tensile membrane stress within the shell varies greatly over the area of the shell surface, reinforcement resisting the total tension may be concentrated in the regions of largest tensile stress where it can be shown that this provides a safe basis for design. However, the ratio of shell reinforcement in any portion of the tensile zone shall be not less than 0.0035 based on the overall thickness of the shell.

19.4.9 Reinforcement required to resist shell bending moments shall be proportioned with due regard to the simultaneous action of membrane axial forces at the same location. Where shell reinforcement is required in only one face to resist bending moments, equal amounts shall be placed near both surfaces of the shell even though a reversal of bending moments is not indicated by the analysis.

19.4.10 Shell reinforcement in any direction shall not be spaced further apart than 500 mm or five times the shell thickness. Where the principal membrane tensile stress on the gross concrete area due to factored loads exceeds $\phi \sqrt{f'_c}/3$ reinforcement shall not be spaced further apart than three times the shell thickness.

19.4.11 Shell reinforcement at the junction of the shell and supporting members or edge members shall be anchored in or extended through such members in accordance with the requirements of Chapter 12, except that the minimum development length shall be 1.2 l_d but not less than 500 mm.

19.4.12 Splice development lengths of shell reinforcement shall be governed by the provisions of Chapter 12, except that the minimum splice length of tension bars shall be 1.2 times the value required by Chapter 12 but not less than 500 mm. The number of splices in principal tensile reinforcement shall be kept to a practical minimum. Where splices are necessary they shall be staggered at least l_d with not more than one-third of the reinforcement spliced at any section.

19.5 Construction

19.5.1 When removal of formwork is based on a specific modulus of elasticity of concrete because of stability or deflection considerations, the value of the modulus of elasticity E_c shall be determined from flexural tests of field-cured beam specimens. The number of test specimens, the dimensions of test beam specimens, and test procedures shall be specified by the Engineer.

19.5.2 The Engineer shall specify the tolerances for the shape of the shell. If construction results in deviations from the shape greater than the specified tolerances, an analysis of the effect of the deviations shall be made and any required remedial actions shall be taken to ensure safe behaviour.

PART 2

STRUCTURAL DESIGN REQUIREMENTS

SECTION 6F

REINFORCED AND PRE-STRESSED CONCRETE

SPECIAL CONSIDERATIONS

PART 2
SECTION 6F

SUB-SECTION 20 – STRENGTH EVALUATION OF EXISTING STRUCTURES

20.1 Strength evaluation - General

If doubt develops concerning the safety of a structure or member, the Building Official may order a structural strength investigation by analyses, or by means of loads tests, or by a combination of analyses and load tests. (For approval of special systems of design or construction see Article 1.4).

20.2 Analytical investigations - General

20.2.1 If strength evaluation is by analysis, a through field investigation shall be made of dimensions and details of members, properties of materials, and other pertinent conditions of the structure as actually built.

20.2.2 Analyses based on investigation required by Article 20.2.1 shall satisfy the Building Official that the load factors meet requirements and intent of this Code. See Article 20.6.

20.3 Load tests - General

20.3.1 If strength evaluation is by load tests, a qualified engineer acceptable to the Building Official shall control such tests.

20.3.2 A load test shall not be made until that portion of the structure to be subject to load is at least 56 days old. When the owner of the structure, the Contractor, and all involved parties mutually agree, the test may be made at an earlier age.

20.3.3 When only a portion of the structure is to be load tested, the questionable portion shall be load tested in such a manner as to adequately test the suspected source of weakness.

20.3.4 Forty-eight hours prior to application of test load, a load to simulate effect of that portion of the dead loads not already acting shall be applied and shall remain in place until all testing has been completed.

20.4 Load tests of flexural members

20.4.1 When flexural members, including beams and slabs, are load tested, the additional provisions of Article 20.4 shall apply.

20.4.2 Base readings (datum for deflection measurements) shall be made immediately prior to application of test load.

20.4.3 That portion of the structure selected for loading shall be subject to a total load, including dead loads already acting, equivalent to 0.85(1.4D + 1.7 L). Determination of L shall include live load reductions as permitted by the General Building Code of which this Code forms a part.

20.4.4 Test load shall be applied in not less than four approximately equal increments without shock to the structure and in such a manner as to avoid arching of loading materials.

20.4.5 After test load has been in position for 24 hr, initial deflection readings shall be taken.

20.4.6 Test load shall be removed immediately after initial deflection readings, and final deflection readings shall be taken 24 hr after removal of the test load.

20.4.7 If the portion of the structure tested shows visible evidence of failure, the portion tested shall be considered to have failed the test and no retesting of the previously tested portion shall be permitted.

20.4.8 If the portion of the structure tested shows no visible evidence of failure, the following criteria shall be taken as indication of satisfactory behaviour:

(a) If measured maximum deflection, a, of a beam, floor or roof is less than $l_t^2/20{,}000$ h.

(b) If measured maximum deflection, a, of a beam, floor, or roof exceeds $l_t^2/20{,}000$ h, deflection recovery within 24 hr after removal of the test load shall be at least 75 percent of the maximum deflection for non-pre-stressed concrete, or 80 percent for pre-stressed concrete.

20.4.9 In Articles 20.4.8(a) and (b), l_t for cantilevers shall be taken as two times the distance from support to cantilever end, and deflection shall be adjusted for any support movement.

20.4.10 Non-pre-stressed concrete construction failing to show 75 percent recovery of deflection as required by Article 20.4.8(b) may be retested not earlier than 72 hr after removal of the first test load. The portion of the structure tested shall be considered satisfactory if:

(a) The portion of the structure tested shows no visible evidence of failure in the retest, and

(b) Deflection recovery caused by second test load is at least 80 percent of the maximum deflection in the second test.

20.4.11 Pre-stressed concrete construction shall not be retested.

20.5 Members other than flexural members

Members other than flexural members preferably shall be investigated by analysis.

20.6 Provisions for lower load rating

If structure under investigation does not satisfy conditions or criteria of Articles 20.2, 20.4.8 or 20.4.10, the Building Official may approve a lower load rating for that structure based on results of the load test or analysis.

20.7 Safety

20.7.1 Load test shall be conducted in such a manner as to provide for safety of life and structure during the test.

20.7.2 No safety measures shall interfere with load test procedures or affect results.

APPENDIX A

SPECIAL PROVISIONS FOR SEISMIC DESIGN

APPENDIX A - SPECIAL PROVISIONS FOR SEISMIC DESIGN

A.0 NOTATION - SEE APPENDIX C

A.1 Definitions

BASE OF STRUCTURE - Level at which earthquake motions are assumed to be imparted to a building. This level does not necessarily coincide with the ground level.

BOUNDARY MEMBERS - Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary members do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms may also have to be provided with boundary members.

COLLECTOR ELEMENTS - Elements that serve to transmit the inertial forces within the diaphragms to members of the lateral-force resisting systems.

CROSS-TIE - A continuous bar having a 135 deg hook with at least a ten-diameter extension at one end and a 90 deg hook with at least a six-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars.

DESIGN LOAD COMBINATIONS - Combinations of factored loads and forces specified in Article 9.2.

DEVELOPMENT LENGTH FOR A BAR WITH A STANDARD HOOK - The shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90 deg hook.

FACTORED LOADS AND FORCES - Specified loads and forces modified by the factors in Article 9.2.

HOOP - A closed tie or continuously wound tie the ends of which have 135 deg hooks with ten-diameter extensions, that encloses the longitudinal reinforcement.

LATERAL-FORCE RESISTING SYSTEM - That portion of the structure composed of members proportioned to resist forces related to earthquake effects.

LOW-DENSITY-AGGREGATE CONCRETE - "All-low-density" or "sanded-low-density" aggregate made with low-density aggregates conforming to Article 3.3.

SHELL CONCRETE - Concrete outside the transverse reinforcement confining the concrete

SPECIFIED LATERAL FORCES - Lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by the governing code for earthquake resistant design.

STRUCTURAL DIAPHRAGMS - Structural members, such as floor and roof slabs, which transmit inertial forces to lateral-force resisting members.

STRUCTURAL TRUSSES - Assemblages of reinforced concrete members subjected primarily to axial forces.

STRUCTURAL WALLS - Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions.

STRUT - An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

TIE ELEMENTS - Elements which serve to transmit inertia forces and prevent separation of such building components as footings and walls.

A.2 General requirements

A.2.1 SCOPE

A.2.1.1 Appendix A contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.

A.2.1.2 The provisions of Chapters 1 through 18 shall apply except as modified by the provisions of this Appendix.

A.2.1.3 In regions of moderate seismic risk, reinforced concrete frames resisting forces induced by earthquake motions shall be proportioned to satisfy only Article A.9 of Appendix A in addition to the requirements of Chapters 1 through 18.

A.2.1.4 In regions of high seismic risk, all structural reinforced concrete members shall satisfy Articles A.2 through A.8 of Appendix A in addition to the requirements of Chapters 1 through 17.

A.2.1.5 A reinforced concrete structural system not satisfying the requirements of this Appendix may be used if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this Appendix.

A.2.2 ANALYSIS AND PROPORTIONING OF STRUCTURAL MEMBERS

A.2.2.1 The interaction of all structural and nonstructural members which materially affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

A.2.2 Rigid members assumed not to be a part of the lateral force resisting system may be used provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members which are not a part of the lateral-force resisting system shall also be considered.

A.2.2.3 Structural members below base of structure required to transmit to the foundation forces resulting from earthquake effects shall also comply with the requirements of Appendix A.

A.2.2.4 All structural members assumed not to be part of the lateral force resisting system shall conform to Article A.8.

A.2.3 STRENGTH REDUCTION FACTORS

Strength reduction factors shall be as given in Article 9.3 except for the following:

A.2.3.1 Except for determining the strength of joints, the shear strength reduction factor shall be 0.6 for any structural member if its nominal shear strength is less than the shear corresponding to development of its nominal flexural strength for the factored-load combinations including earthquake effect. Shear strength reduction factor for joints shall be 0.85.

A.2.3.2 The strength reduction factor for axial compression and flexure shall be 0.5 for all frame members with factored axial compressive forces exceeding $(A_g f'_c/10)$ if the transverse reinforcement does not conform to Article A.4.4.

A.2.4 CONCRETE IN MEMBERS RESISTING EARTHQUAKE-INDUCED FORCES

A.2.4.1 Compressive strength f'_c of the concrete shall be not less than 20 MPa.

A.2.4.2 Compressive strength of low-density-aggregate concrete used in design shall not exceed 30 MPa. Low-density-aggregate concrete with higher design compressive strength may be used if demonstrated by experimental evidence that structural members made with that low-density-aggregate concrete provide strength and toughness equal to or exceeding those of comparable members made with normal density-aggregate concrete of the same strength.

A.2.5 REINFORCEMENT IN MEMBERS RESISTING EARTHQUAKE INDUCED FORCES

A.2.5.1 Reinforcement resisting earthquake-induced flexural and axial forces in frame members and in wall boundary members shall comply with ASTM A 706. Reinforcement required by design load combinations which include earthquake effect shall not be welded except as specified in Articles A.3.2.4 and A.4.3.2.

A.3 Flexural members of frames

A.3.1 SCOPE

Requirements of Article A.3 apply to frame members (a) resisting earthquake-induced forces (b) proportioned primarily to resist flexure, and (c) satisfying the following conditions:

A.3.1.1. Factored axial compressive force on the member shall not exceed $(A_g f'_c/10)$.

A.3.1.2 Clear span for the member shall not be less than four times its effective depth.

A.3.1.3 The width-to-depth ratio shall not be less than 0.3.

A.3.1.4 The width shall not be less than (a) 250 mm and (b) more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three-fourths of the depth of the flexural member.

A.3.2 LONGITUDINAL REINFORCEMENT

A.3.2.1 At any section of a flexural member and for the top as well as for the bottom reinforcement the amount of reinforcement shall not be less than $(1.4 b_w d/f_y)$ and the reinforcement ratio ρ shall not exceed 0.025. At least two bars shall be provided continuously both top and bottom.

A.3.2.2 Positive moment-strength at joint face shall be not less than one-half of the negative moment strength provided at that face of the joint. Neither the negative - nor the positive - moment strength at any section along member length shall be less than one-fourth the maximum moment strength provided at face of either joint.

A.3.2.3 Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed d/4 or 100 mm. Lap splices shall not be used (a) within the joints, (b) within a distance of twice the member depth from the face of the joint, and (c) at locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame.

A.3.2.4 Welded splices and mechanical connections conforming to Articles 12.14.3.1 through 12.14.3.4 may be used for splicing provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the centre-to-centre distance between splices of adjacent bars is 600 mm or more measured along the longitudinal axis of the frame member.

A.3.3 TRANSVERSE REINFORCEMENT

A.3.3.1 Hoops shall be provided in the following regions of frame members:

(1) Over a length equal to twice the member depth measured from the face of the supporting member toward mid-span, at both ends of the flexural member.
(2) Over lengths equal to twice the member depth on both sides of a section where flexural yielding may occur in connection with inelastic lateral displacements of the frame.

A.3.3.2 The first hoop shall be located not more than 50 mm from the face of a supporting member. Maximum spacing of the hoops shall not exceed (a) d/4, (b) eight times the diameter of the smallest longitudinal bars, (c) 24 times the diameter of the hoop bars, and (d) 300 mm.

A.3.3.3 Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to Article 7.10.5.3.

A.3.3.4 Where hoops are not required, stirrups shall be spaced at no more than d/2 throughout the length of the member.

A.3.3.5 Hoops in flexural members may be made up of two pieces of reinforcement, a stirrup having 135 deg hooks with ten-diameter extensions anchored in the confined core and a cross-tie to make a closed hoop. Consecutive cross-ties shall have their 90 deg hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the cross-ties are confined by a slab only on one side of the flexural frame member, the 90 deg hooks of the cross-ties shall all be placed on that side.

A.4 Frame members subjected to bending and axial load

A.4.1 SCOPE

The requirements of this section apply to frame members (a) resisting earthquake-induced forces, (b) having a factored axial compressive force exceeding ($A_g f'_c/10$) and (c) satisfying the following conditions:

A.4.1.1 The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 300 mm.

A.4.1.2 The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

A.4.2 MINIMUM FLEXURAL STRENGTH OF COLUMNS

A.4.2.1 Flexural strength of any column proportioned to resist a factored axial compressive force exceeding ($A_g f'_c/10$) shall satisfy Article A.4.2.2 or A.4.2.3.

Lateral strength and stiffness of columns not satisfying Article A.4.2.2. shall be ignored in determining the calculated strength and stiffness of the structure but shall conform to Article A.8.

A.4.2.2. The flexural strengths of the columns shall satisfy Eq. (A-1)

$$\sum M_e \geq (6/5)\sum M_g \qquad \text{(A-1)}$$

$\sum M_e$ = sum of moments, at the centre of the joint, corresponding to the design flexural strength of the columns framing into that joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

$\sum M_g$ = sum of moments, at the centre of the joint, corresponding to the design flexural strengths of the girders framing into that joint.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Eq. (A-1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

A.4.2.3 If Article A.4.2.2 is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in Article A.4.4 over their full height.

A.4.3 LONGITUDINAL REINFORCEMENT

A.4.3.1 The reinforcement ratio, , shall not be less than 0.01 and shall not exceed 0.06.

A.4.3.2 Lap splices are permitted only within the centre half of the member length and shall be proportioned as tension splices. Welded splices and mechanical connections conforming to Articles 12.14.3.1 through 12.14.3.4 may be used for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at a section and the distance between splices is 600 mm or more along the longitudinal axis of the reinforcement.

A.4.4 TRANSVERSE REINFORCEMENT

A.4.4.1 Transverse reinforement as specified below shall be provided unless a larger amount is required by Article A.7.

(1) The volumetric ratio of spiral or circular hoop reinforcement ρ_s, shall not be less than that indicated by Eq. (A-2)

$$\rho_s = 0.12f'_c/f_{yh} \qquad \text{(A-2)}$$

and shall not be less than that required by Eq. (10-5)

(2) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by Eq. (A-3) and (A-4).

$$A_{sh} = 0.3\ (sh_c f'_c/f_{yh})((A_g/A_{ch}) - 1) \qquad \text{(A-3)}$$

$$A_{sh} = 0.12sh_c f'_c/f_{yh} \qquad \text{(A-4)}$$

(3) Transverse reinforcement may be provided by single or overlapping hoops. Cross-ties of the same bar size and spacing as the hoops may be used. Each end of the cross-ties shall engage a peripheral longitudinal reinforcing bar. Consecutive cross-ties shall be alternated end for end along the longitudinal reinforcement.

(4) If the design strength of member core satisfies the requirement of the specified loading combinations including earthquake effect, Eq. (A-3) and (10-5) need not be satisfied.

A.4.4.2 Transverse reinforcement shall be spaced at distances not exceeding (a) one-quarter of the minimum member dimension and (b) 100 mm.

A.4.4.3 Cross-ties or legs of overlapping hoops shall not be spaced more than 350 mm on centre in the direction perpendicular to the longitudinal axis of the structural member.

A.4.4.4 Transverse reinforcement in amount specified in Articles A.4.4.1 through A.4.4.3 shall be provided over a length l_o from each joint face and on both sides of any section where flexural yielding may occur in connection with inelastic lateral displacements of the frame. The length l_o shall not be less than (a) the depth of the member at the joint face or at the section where flexural yielding may occur, (b) one-sixth of the clear span of the member, and (c) 500 mm.

A.4.4.5 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as specified in Articles A.4.4.1 through A.4.4.3 over their full height beneath the level at which the discontinuity occurs if the factored axial compressive force in these members, related to earthquake effect, exceeds $(A_g f'_c/10)$.

A.5 Structural walls, diaphragms, and trusses

A.5.1 SCOPE

The requirements of this article apply to structural walls and trusses serving as parts of the earthquake force resisting systems as well as to diaphragms, struts, ties, chords and collector members which transmit forces induced by earthquake.

A.5.2. REINFORCEMENT

A.5.2.1 The reinforcement ratio, ρ_v, for structural walls shall not be less than 0.0025 along the longitudinal and transverse axes. Reinforcement spacing each way shall not exceed 500 mm. Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane.

A.5.2.2 At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $A_{cv} \sqrt{f'_c}/6$.

A.5.2.3 Structural-truss members, struts, ties, and collector members with compressive stresses exceeding $0.2f'_c$ shall have special transverse reinforcement, as specified in Article A.4.4, over the total length of the member. The special transverse reinforcement may be discontinued at a section whee the calculated compressive stress is less than $0.15f'_c$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross-section properties of the members considered.

A.5.2.4 All continuous reinforcement in structural walls, diaphragms, trusses, struts, ties, chords, and collector members shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Article A.6.4.

A.5.3 BOUNDARY MEMBERS FOR STRUCTURAL WALLS AND DIAPHRAGMS

A.5.3.1 Boundary members shall be provided at boundaries sadn edges around openings of structural walls and diaphragms for which the maximum extreme-fiber stress, corresponding to factored forces including earthquake effect, exceeds $0.2f'_c$ unless the entire wall or diaphragm member is reinforced to satisfy Articles A.4.4.1 through A.4.4.3. The boundary member may be discontinued where the calculated compressive stress is less than $0.15f'_c$. Stresses shall be calculated for the factored forces using a linerly elastic model and cross-section properties.

A.5.3.2 Boundary members, where required, shall have transverse reinforcement as specified in Articles A.4.4.1 through A.4.4.3.

A.5.3.3 Boundary members of structural walls shall be proportioned to carry all factored gravity loads on the wall, including tributary loads and self-weight, as well as the vertical force required to resist overturning moment calculated from factored forces related to earthquake effect.

A.5.3.4 Boundary members of structural diaphragms shall be proportioned to resist the sum of the compressive force acting in the plane of the diaphragm and the force obtained from dividing the factored moment at the section by the distance between the edges of the diaphragm at that section.

A.5.3.5 Transverse reinforcement in walls, with boundary members shall be anchored within the confined core of the boundary member to develop the yield stress in tension of the transverse reinforcement.

A.5.4 CONSTRUCTION JOINTS

A.5.4.1 All construction joints in walls and diaphragms shall conform to Article 6.4 and contact surfaces shall be roughened as specified in Article 11.7.9.

A.6 Joints of frames

A.6.1 GENERAL REQUIREMENTS

A.6.1.1 Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is $1.25f_y$.

A.6.1.2 Strength of the joint shall be governed by the appropriate strength reduction factors specified in Article 9.3.

A.6.1.3 Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to Article A.6.4 and in compression according to Chapter 12.

A.6.2 TRANSVERSE REINFORCEMENT

A.6.2.1 Transverse hoop reinforcement, as specified in Article A.4.4 shall be provided within the joint, unless the joint is confined by structural members as specified in Article A.6.2.2.

A.6.2.2 Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by Article A.4.4.1 shall be provided where members frame into all four sides of the joint and where each member width is at least three-fourths the columns width.

A.6.2.3 Transverse reinforement as required by Article A.4.4 shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

A.6.3 SHEAR STRENGTH

A.6.3.1 The nominal shear strength of the joint shall be assumed not to exceed the forces specified below for normal density-aggregate concrete.

For confined joint	$1.7\sqrt{f'_c}A_j$
For others	$1.5\sqrt{f'_c}A_j$

where A_j is the minimum cross-sectional area of the joint in a plane parallel to the axis of the reinforcement generating the shear force.

A joint is considered to be confined if members frame into all vertical faces of the joint and if at least three-quarters of each face of the joint is covered by the framing member.

A.6.3.2 For low-density-aggregate concrete, the nominal shear strength of the joint shall not exceed three-quarters of the limits given in Article A.6.3.1.

A.6.4 DEVELOPMENT LENGTH OF BARS IN TENSION

A.6.4.1 The development length, l_{dh}, for a bar with a standard 90-deg hook in normal density-aggregate concrete shall not be less than $8d_b$, 150 mm, and the length required by Eq. (A-5).

$$l_{dh} = f_y d_b / 5.4 \sqrt{f'_c} \qquad \text{(A-5)}$$

for bar sizes through number 10 and number 35.

For low-density-aggregate concrete, the development length for a bar with a standard 90-deg hook shall not be less than 10 d_b, 190 mm, and 1.25 times that required by Eq. (A-5).

The 90-deg hook shall be located within the confined core of a column or of a boundary member.

A.6.4.2 For bar sizes no. 10 through no. 35, the development length, l_d, for a straight bar shall not be less than (a) two-and-a-half (2.5) times the length required by Article A.6.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm and (b) three-and-a-half (3.5) times the length required by Article A.6.4.1 if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.

A.6.4.3 Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary member. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

A.7 **Shear-strength requirements**

A.7.1 DESIGN FORCES

A.7.1.1 Frame members subjected primarily to bending

The design shear force, V_e, shall be determined from consideration of the statical forces on the portion of the member between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable strength act at the joint faces and that the member is loaded with the factored tributary gravity load along its span. The moments corresponding to probable strength shall be calculated using the properties of the member at the joint faces without strength reduction factors and assuming that the stress in the tensile reinforcement is equal to at least $1.25f_y$.

A.7.1.2 Frame members subjected to combined bending and axial load

The design shear force, V_e, shall be determined from consideration of the forces on the member, with the nominal moment strengths calculated for the factored axial compressive force resulting in the largest moment, acting at the faces of the joints.

A.7.1.3 Structural walls, diaphragms and trusses

The design shear force, V_u, shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in Article 9.2.

A.7.2 TRANSVERSE REINFORCEMENT IN FRAME MEMBERS

A.7.2.1 For determining the required transverse reinforcement in frame members in which the earthquake-induced shear force calculated in accordance with Article A.7.1.1 represent one-half or more of total design shear, the quantity V_c shall be assumed to be zero of the factored axial compressive force including earthquake effects is less than $(A_g f'_c/20)$.

A.7.2.2 Stirrups or ties required to resist shear shall be hoops over lengths of members as specified in Articles A.3.3, A.4.4 and A.6.2.

A.7.3 SHEAR STRENGTH OF STRUCTURAL WALLS AND DIAPHRAGMS

A.7.3.1 Nominal shear strength of structural walls and diaphragms shall be determined using either Article A.7.3.2 or A.7.3.3.

A.7.3.2 Nominal shear strength, V_n, of structural walls and diaphragms shall be assumed not to exceed the shear force calculated from

$$V_n = A_{cv}\left[\left(\sqrt{f'_c}/6 + \rho_n f_y\right)\right] \qquad \text{(A-6)}$$

A.7.3.3 For walls (diaphragms) and wall (diaphragm) segments having a ratio of (h_w/l_w) less than 2.0, nominal shear strength of wall (diaphragm) may be determined from Eq. (A-7).

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_n f_y) \qquad \text{(A-7)}$$

where the coefficient α_c varies linearly from 1/4 for $(h_w/l_w) = 1.5$ to 1/6 for $(h_w/l_w) = 2.0$

A.7.3.4 In Article A.7.3.3, value of ratio (h_w/l_w) used for determining V_n for segments of a wall or diaphragm shall be the larger of the ratios for the entire wall (diaphragm) and the segment of wall (diaphragm) considered.

A.7.3.5 Walls (diaphragms) shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall (diaphragm). If the ratio (h_w/l_w) does not exceed 2.0, reinforcement ratio, ρ_v shall not be less than reinforcement ratio ρ_n.

A.7.3.6 Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed $2A_{cv}\sqrt{f'_c}/3$ where A_{cv} is the total cross-sectional area and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed $5A_{cp}\sqrt{f'_c}/6$ where A_{cp} represents the cross-sectional area of the pier considered.

A.7.3.7 Nominal shear strength of horizontal wall segments shall not be assumed to exceed $5A_{cp}\sqrt{f'_c}/6$ where A_{cp} represents the cross-sectional area of a horizontal wall segment.

A.8 Frame members not proportioned to resist forces induced by earthquake motions

A.8.1 All frame members assumed not to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load carrying capacity with the structure assumed to have deformed laterally twice that calculated for the factored lateral forces. Such member shall satisfy the minimum-reinforcement requirements specified in Articles A.3.2.1 and A.5.2.1 as well as those specified in Chapter 7, 10 and 11.

A.8.2 All frame members with factored axial compressive forces exceeding$(A_g f'_c/10)$ shall satisfy the following special requirements unless they comply with Article A.4.4.

A.8.2.1 Ties shall have 135-deg hooks with extensions not less than six tie-bar diameters or 60 mm. Cross-ties, as defined in this Appendix, may be used.

A.8.2.2 The maximum tie spacing shall be s_o over a length l_o measured from the joint face. The spacing s_o shall be not more than (a) eight diameters of the smallest longitudinal bar enclosed, (b) 24 tie-bar diameters, and (c) one-half the least cross-sectional dimension of the column. The length l_o shall not be less than (a) one-sixth of the clear height of the column, (b) the maximum cross-sectional dimension of the column, and (c) 500 mm.

A.8.2.3 The first shall be within a distance equal to $0.5s_o$ from the face of the joint.

A.8.2.4 The tie spacing shall not exceed $2s_o$ in any part of the column.

A.9 Requirements for frames in regions of moderate seismic risk.

A.9.1 In regions of moderate seismic risk, structural frames proportioned to resist forces induced by earthquake motions shall satisfy the requirements of Article A.9 in addition to those of Chapter 1 through 18.

A.9.2 Reinforcement details in a frame member shall satisfy Article A.9.4 if the factored compressive axial load for the member does not exceed ($A_g f'_c/10$). If the factored compressive axial load is larger, frame reinforcement details shall satisfy Article A.9.5 unless the member has spiral reinforcement according to Eq. (10-5). If a two-way slab system without beams is treated as part of a frame resisting earthquake effect, reinforcement details in any span resisting moments caused by lateral force shall satisfy Article A.9.6.

A.9.3 Design shear strength of beams, columns, and two-way slabs resisting earthquake effect shall not be less than either (a) the sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads or (b) the maximum shear obtained from design load combinations which include earthquake effect E, with E assumed to be twice that prescribed by the governing code for earthquake-resistant design.

A.9.4 BEAMS

A.9.4.1 The positive-moment strength at the face of the joint shall be not less than one-third the negative moment strength provided at that face of the joint. Neither the negative- nor the positive-moment strength at any section along the length of the member shall be less than one-fifth the maximum moment strength provided at the face of either joint.

A.9.4.2 At both ends of the member, stirrups shall be provided over lengths equal to twice the member depth measured from the face of the supporting member toward mid-span. The first stirrup shall be located at not more than 500 mm from the face of the supporting member. Maximum stirrup spacing shall not exceed (a)d/4, (b) eight times the diameter of the smallest longitudinal bar enclosed, (c) 24 times the diameter of the stirrup bar, and (d) 300 mm.

A.9.4.3 Stirrups shall be placed at not more than d/2 throughout the length of the member.

A.9.5 COLUMNS

A.9.5.1 Maximum tie spacing shall not exceed s_o over a length l_o measured from the joint face. Spacing s_o shall not exceed (a) eight times the diameter of the smallest longitudinal bar enclosed, (b) 24 times the diameter of the tie bar, (c) one-half of the smallest cross-sectional dimension of the frame member, and (d) 300 mm. Length l_o shall not be less than (a) one-sixth of the clear span of the member, (b) maximum cross-sectional dimension of the member, and (c) 500 mm.

A.9.5.2 The first tie shall be located at not more than $s_o/2$ from the joint face.

A.9.5.3 Joint reinforcement shall conform to Article 11.12.1.2.

A.9.5.4 Tie spacing shall not exceed twoce the spacing s_o.

A.9.6 TWO-WAY SLABS WITHOUT BEAMS

A.9.6.1 Factored slab moment at support related to earthquake effect shall be determined for load combinations defined by Eq. (9-2) and (9-3). All reinforcement provided to resist M_s, the portion of slab moment balanced by support moment, shall be placed within the column strip defined in Article 13.2.1.

A.9.6.2 The fraction, defined by Eq. (13-1), or moment M_s shall be resisted by reinforcement placed within the effective width specified in Article 13.3.3.2.

A.9.6.3 Not less than one-half of the reinforcement in the column strip at support shall be placed within the effective slab width specified in Article 13.3.3.2.

A.9.6.4 Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.

A.9.6.5 Continuous bottom reinforcement in the columns strip shall be not less than one-third of the top reinforcement at the support in the column strip.

A.9.6.6 Not less than one-half of all bottom reinforcement at mid-span shall be continous and shall develop its yield strength at face of support as defined in Article 13.6.2.5.

A.9.6.7 At discontinous edges of the slab all top and bottom reinforcement at support shall be developed at the face of support as defined in Article 13.6.2.5.

A.10 Cantilevered and coupled shear walls

A.10.1 In ductile structures where lateral earthquake laods are resisted by a system of cantilever or coupled shear walls allowance shall be made for the dynamic magnification of shear forces. (See Reference).

Also, the energy dissipation of all types of shear walls will be mainly through flexural yielding. "Squat" Shear walls should be detailed to behave as ductile resisting system.

Ref. NZS 4203 - Shear Wall Structures

A.10.2 All walls to which lateral earthquake load is assigned shall be designed to be capable of dissipating seismic energy by flexural yielding.

A.10.3 Appropriately modified capacity design procedures shall be used to ensure that the ideal shear strength of walls is in excess of the shear force when flexural over-strength is reached.

A.10.4 When two or more cantilever walls are interconnected in the same plane at intervals by substantial ductile beams, part of the seismic energy to be dissipated shall be assigned to the coupling system. Capacity design procedures shall be used to ensure that the energy dissipation in the coupling system can be maintained at its flexural overstrength.

APPENDIX B

ALTERNATE DESIGN METHOD

APPENDIX B - ALTERNATE DESIGN METHOD

B.0 Notation

Some notation definitions are modified from that in the main body of the Code for specific use in the application of Appendix B.

A_g gross area of section, mm^2

A_1 loaded area

A_2 maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area

A_v area of shear reinforcement within a distance s, mm^2

b width of compression face of member, mm

b_o perimeter of critical section for slabs and footings, mm

b_w web width, or diameter of circular section, mm

d distance from extreme compression fibre to centroid of tension reinforcement

E_c modulus of elasticity of concrete, MPa. See Article 8.5.1

E_s modulus of elasticity of reinforcement, MPa. See Article 8.5.2

f'_c specified compressive strength of concrete, MPa. See Chapter 4

$\sqrt{f'_c}$ square root of specified compressive strength of concrete, MPa

f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa. See Article 4.1.4

f_s permissible tensile stress in reinforcement, MPa

f_y specified yield strength of reinforcement, MPa. See Article 3.5.3

M design moment

n modular ratio of elasticity E_s/E_c

N design axial load normal to cross-section occurring simultaneously with V; to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage

s spacing of shear reinforcement in direction parallel to longitudinal reinforcement, mm

v design shear stress

v_c permissible shear stress carried by concrete, MPa

v_h permissible horizontal shear stress, MPa

V design shear force at section

α angle between inclined stirrups and longitudinal axis of member

β_c ratio of long side to short side of concentrated load or reaction area

ρ ratio of tension reinforcement A_s/bd

ϕ strength reduction factor. See Article B.2.1

B.1 Scope

B.1.1 Non-pre-stressed reinforced concrete members may be designed using service loads (without load factors) and permissible service load stresses in accordance with provisions of Appendix B.

B.1.2 For design of members not covered by Appendix B, appropriate provisions of this Code shall apply.

B.1.3 All applicable provisions of this Code for non-pre-stressed concrete, except Article 8.4, shall apply to members designed by the Alternate Design Method.

B.1.4 Flexural members shall meet requirements for deflection control in Article 9.5, and requirements of Articles 10.4 through 10.7 of this Code.

B.2 General

B.2.1 Load factors and strength reduction factors ϕ shall be taken as unity for members designed by the Alternate Design Method.

B.2.2 Members may be proportioned for 75 percent of capacities required by other parts of Appendix B when considering wind or earthquake forces combined with other loads, provided the resulting section is not less than that required for the combination of dead and live load.

B.2.3 When dead load reduces effects of other loads, members shall be designed for 85 percent of dead load in combination with the other loads.

B.3 Permissible service load stresses

B.3.1 Stresses in concrete shall not exceed the following:

(a) Flexure
Extreme fibre stress
in compression........................$0.45\ f'_c$

(b) Shear*
Beams and one-way slabs
and footings:
Shear carried by concrete, v_c ....$f'_c/11$
Maximum shear carried by
concrete plus shear
reinforcement .... $v_c + 3\ \ f'_c/8$
Joists**
Shear carried by concrete,
v_c .... $f'_c/10$
Two-way slabs and footings:
Shear carried by concrete,
v_c***...................... $1 + 2 \quad f'_c/12$
c

but not greater than $f'_c/6$

(c) Bearing on loaded area****................. $0.3f'_c$

* For more detailed calculation of shear stress carried by concrete v_c and shear values for low-density aggregate concrete, see Article B.7.4

** Designed in accordance with Article 8.11 of this Code.

*** If shear reinforcement is provide see Article B.7.7.4 and B.7.7.5

**** When the supporting surface is wider on all sides than the loaded area, permissible bearing stress on the loaded area may be multiplied by $\sqrt{A_2/A_1}$ but not more than 2. When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

B.3.2 Tensile stress in reinforcement f_s shall not exceed the following:

(a) Grade 300 reinforcement.......................... 140 MPa

(b) Grade 400 reinforcement or greater and welded wire fabric (smooth or deformed)..............................170 MPa

(c) For flexural reinforcement, 10 mm or less in diameter, in one-way slabs of not more than 4 m span,................................ $0.50\ f_y$ but not greater than 200 MPa

B.4 Development and splices of reinforcement

B.4.1 Development and splices of reinforcement shall be as required in Chapter 12 of this Code.

B.4.2 In satisfying requirements of Article 12.11.3, M_n shall be take as computed moment capacity assuming all positive moment tension reinforcement at the section to be stressed to the permissible tensile stress f_s, and V_u shall be taken as unfactored shear force at the section.

B.5 Flexure

For investigation of stresses at service loads, straight line theory (for flexure) shall be used with the following assumptions.

B.5.1 Strains vary linearly as the distance from the neutral axis, except, for deep flexural members with overall depth-span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of strain shall be considered. See Article 10.7 of this Code.

B.5.2 Stress-strain relationship of concrete is a straight line under service loads within permissible service load stresses.

B.5.3 In reinforced concrete members, concrete resists no tension.

B.5.4 Modular ratio, $n = E_s/E_c$, may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, value of n for low-density concrete shall be assumed to be the same as for normal density concrete of the same strength.

B.5.5 In doubly reinforced flexural members, an effective modular ratio of $2E_s/E_c$ shall be used to transform compression reinforcement for stress computations. Compressive stress in such reinforcement shall not exceed permissible tensile stress.

B.6 Compression members with or without flexure

B.6.1 Combined flexure and axial load capacity of compression members shall be taken as 40 percent of that computed in accordance with provisions in Chapter 10 of this Code.

B.6.2 Slenderness effects shall be included according to requirements of Article 10.10 and 10.11. In Eq. (10-7) and (10-8) the term P_u shall be replaced by 2.5 times the design axial load, and ϕ shall be taken equal to 1.0.

B.6.3 Walls shall be designed in accordance with Chapter 14 of this Code with flexure and axial load capacities taken as 40 percent of that computed using Chapter 14. In Eq. (14-1), ϕ shall be taken equal to 1.0.

B.7 Shear and torsion

B.7.1 Design shear stress v shall be computed by

$$v = \frac{V}{b_w d} \tag{B-1}$$

where V is design shear force at section considered.

B.7.2 When the reaction, in direction of applied shear, introduces compression into the end regions of a member, sections located less than a distance d from face of support may be designed for the same shear v as that computed at a distance d.

B.7.3 Whenever applicable, effects of torsion, in accordance with provisions of Chapter 11 of this Code, shall be added. Shear and torsional moment strengths provided by concrete and limiting maximum strengths for torsion shall be taken as 55 percent of the values given in Chapter 11.

B.7.4 SHEAR STRESS CARRIED BY CONCRETE

B.7.4.1 For members subject to shear and flexure only, shear stress carried by concrete v_c shall not exceed $1.1\sqrt{f'_c}$ unless a more detailed calculation is made in accordance with Article B.7.4.4.

B.7.4.2 For members subject to axial compression, shear stress carried by concrete v_c shall not exceed $\sqrt{f'_c}/11$ unless a more details calculation is made in accordance with Article B.7.4.5.

B.7.4.3 For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using

$$v_c = 1 + 0.6\frac{N}{A_g} \quad f'_c/11 \qquad \text{(B-2)}$$

where N is negative for tension. Quantity N/A_g shall be expressed in MPa.

B.7.4.4 For members subject to shear and flexure only, v_c may be computed by

$$V_c = (\quad f'_c/12) \; + 9 \;_w \frac{Vd}{M} \qquad \text{(B-3)}$$

but v_c shall not exceed $\sqrt{f'_c}/7$. Quantity Vd/M shall not be taken greater than 1.0, where M is design moment occurring simultaneously with V at section considered.

B.7.4.5 For members subject to axial compression, v_c may be computed by

$$v_c = \quad 1 + \frac{N}{10A_g} \qquad f'_c/11 \qquad \text{(B-4)}$$

Quantity N/A_g shall be expressed in MPa.

B.7.4.6 Shear stresses carried by concrete v_c apply to normal density concrete. When low-density aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified and concrete is proportioned in accordance with Article 4.2, $1.8f_{ct}$ shall be substituted for $\sqrt{f'_c}$, but the value of $1.8f_{ct}$ shall not exceed $\sqrt{f'_c}$.

(b) When f_{ct} is not specified, the value of $\sqrt{f'_c}$ shall be multiplied by 0.75 for "all-low-density" concrete and by 0.85 for "sand-low-density" concrete. Linear interpolation may be applied when partial sand replacement is used.

B.7.4.7 In determining shear stress carried by concrete v_c, whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable-depth members may be included.

B.7.5 SHEAR STRESS CARRIED BY SHEAR REINFORCEMENT

B.7.5.1 Types of shear reinforcement

Shear reinforcement may consist of:

(a) Stirrups perpendicular to axis of member

(b) Welded wire fabric with wires located perpendicular to axis of member making an angle of 45 deg or more with longitudinal tension reinforcement

(c) Longitudinal reinformcement with bent portion making an angle of 30 deg or more with longitudinal tension reinforcement

(d) Combinations of stirrups and bent longitudinal reinforcement

(e) Spirals

B.7.5.2 Design yield strength of shear reinforcement shall not exceed 400 MPa.

B.7.5.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fibre and shall be anchored at both ends according to Article 12.13 of this Code to develop design yield strength of reinforcement.

B.7.5.4 Spacing limits for shear reinforcement

B.7.5.4.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed d/2, nor 600 mm.

B.7.5.4.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-deg line, extending toward the reaction from mid-depth of member (d/2) to longitudinal tension reinforcement shall be crossed by at least one line of shear reinforcement.

B.7.5.4.3 When $(v - v_c)$ exceeds $\sqrt{f'_c}/6$, maximum spacing given in Articles B.7.5.4.1 and B.7.5.4.2 shall be reduced by one-half.

B.7.5.5 Minimum shear reinforcement

B.7.5.5.1 A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members where design shear stress v is greater than one-half the permissible shear stress v_c carried by concrete, except:

(a) Slabs and footings

(b) Concrete joist construction defined by Article 8.11 of this Code.

(c) Beams with total depth not greater than 250 mm, 2-1/2 times thickness of flange, or one-half the width of web, whichever is greatest.

B.7.5.5.2 Minimum shear reinforcement requirements of Article B.7.5.5.1 may be waived if shown by test that required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

B.7.5.5.3 Where shear reinforcement is required by Article B.7.5.5.1 or by analysis, minimum area of shear reinforcement shall be computed by

$$A_v = \frac{b_w s}{3 f_y} \qquad \text{(B-5)}$$

where b_w and s are in millimetres.

B.7.5.6 Design of shear reinforcement

B.7.5.6.1 Where design shear stress v exceeds shear stress carried by concrete v_c, shear reinforcement shall be provided in accordance with Articles B.7.5.6.2 through B.7.5.6.8.

B.7.5.6.2 When shear reinforcement perpendicular to axis of member is used,

$$A_v = \frac{(v - v_c) b_w s}{f_s} \qquad \text{(B-6)}$$

B.7.5.6.3 When inclined stirrups are used as shear reinforcement,

$$A_v = \frac{(v - v_c) b_w s}{f_s(\sin \quad + \cos \quad)} \qquad \text{(B-7)}$$

B.7.5.6.4 When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support,

$$A_v \quad \frac{(v - v_c) b_w d}{f_s \sin} \qquad \text{(B-8)}$$

where $(v - v_c)$ shall not exceed $\sqrt{f'_c}/8$.

B.7.5.6.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, required area shall be computed by Eq. (B-7).

B.7.5.6.6 Only the centre three-quarters of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

B.7.5.6.7 When more than one type of shear reinforcement is used to reinforce the same portion of a member, required area shall be computed as the sum of the various types separately. In such computations, v_c shall be included only once.

B.7.5.6.8 Value of $(v - v_c)$ shall not exceed $3\sqrt{f'_c}/8$.

B.7.6 SHEAR-FRICTION

Where it is appropriate to consider shear transfer across a given plane such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times, shear-friction provisions of Article 11.7 of this Code may be applied, with limiting maximum stress for shear taken as 55 percent of that given in Article 11.7.5. Permissible stress in shear-friction reinforcement shall be that given in Article B.3.2.

B.7.7 SPECIAL PROVISIONS FOR SLABS AND FOOTINGS

B.7.7.1 Shear capacity of slabs and footings in the vicinity of concentrated loads or reactions is governed by the more severe of two conditions:

B.7.7.1.1 Beam action for slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles B.7.1 through B.7.5.

B.7.7.1.2 Two-way action for slab or footing, with a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than d/2 to perimeter of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles B.7.7.2 and B.7.7.3.

B.7.7.2 Design shear stress v shall be computed by

$$v = \frac{V}{b_o d} \qquad \text{(B-9)}$$

where V and b_o shall be taken at the critical section defined in Article B.7.7.1.2.

B.7.7.3 Design shear stress v shall not exceed v_c given by Eq. (B-10) unless shear reinforcement is provided.

$$v_c = 1 + \frac{2}{B_c} \quad f'_c/12 \qquad \text{(B-10)}$$

but v_c shall not exceed $\sqrt{f'_c}/6$. β_c is the ratio of long side to short side of concentrated load or reaction area. When low-density aggregate concrete is used, the modifications of Article B.7.4.6 shall apply.

B.7.7.4 If shear reinforcement consisting of bars or wires is provided in accordance with Article 11.11.3 of this Code, v_c shall not exceed $\sqrt{f'_c}/12$, and v shall not exceed $\sqrt{f'_c}/4$.

B.7.7.5 If shear reinforcement consisting of steel I-or channel-shaped sections (shearheads) is provided in accordance with Article 11.11.4 of this Code, v on the critical section defined in Article B.7.7.1.2 shall not exceed $0.3\sqrt{f'_c}$, and v on the critical section defined in Article 11.11.4.7 shall not exceed $\sqrt{f'_c}/6$. In Eq. (11-38) and (11-39), design shear force V shall be multiplied by 2 and substituted for V_u.

B.7.8 SPECIAL PROVISIONS FOR OTHER MEMBERS

For design of deep flexural members, brackets and corbels, and walls, the special provisions of Sub-Section 11 of this Code shall be used, with shear strengths provided by concrete and limiting maximum strengths for shear taken as 55 percent of the values given in Sub-Section 11. In Article 11.10.6, the design axial load shall be multiplied by 1.2 if compression and 2.0 if tension, and substituted for N_u.

B.7.9 COMPOSITE CONCRETE FLEXURAL MEMBERS

For design of composite concrete flexural members, permissible horizontal shear stress v_h shall not exceed 55 percent of the horizontal shear strengths given in Article 17.5.2 of this Code.

APPENDIX C

NOTATION

APPENDIX C - NOTATION

a	depth of equivalent rectangular stress block as defined in Article 10.2.7. Sub-section 10 and 12.
a	shear span, distance between concentrated load and face of support. Sub-section 11.
a	maximum deflection under test load of member relative to a line joining the ends of the span, or of the free end of a cantilever relative to its support, mm. Sub-section 20.
a	deflection at x. Sub-section 9.
A	effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires, mm^2. When the flexural reinforcement consists of different bar or wire sizes the numbers of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used. Sub-section 10.
A	area of that part of cross-section between flexural tension face and center of gravity of gross- section, mm^2. Sub-section 18
A_b	area of individual bar, mm^2. Sub-section 12
A_c	area of core of spirally reinforced compression member measured to outside diameter of spiral, mm^2. Sub-section 10
Ac	area of concrete section resisting shear transfer. Sub-section 11
A_c	area of contact surface being investigated for horizontal shear, mm^2. Sub-section 17
A_c	area of concrete at cross-section considered, mm^2. Sub-section 18
A_{ch}	cross-sectional ara of a structural member measured out-to-out of transverse reinforcement, mm^2. Appendix A

A_{cp} — area of concrete section, resisting shear, of an individual pier or horizontal wall segment, mm^2. Appendix A

A_{cv} — net area of concrrete section bounded by web thickness and length of section in the direction of shear force considered, mm^2. Appendix A

A_f — area of reinforcement in bracket or corbel resisting factored moment. $(V_{ua} + N_{uc}(H - d))$, mm^2 Sub-section 11

A_g — gross area of section, mm^2. Sub-sections 9, 10, 11, 14, and Appendixes A and B

A_h — area of shear reinforcement parallel to flexural tension reinforcement, mm^2. Sub-section 11

A_j — minimum cross-sectional area within a joint in a plane parallel to the axis of the reinforcement generating the shear in the joint. Where a girder frames into a support of larger width, effective width of the joint shall be assumed not to exceed the width plus the overall depth of the joint, mm^2. Appendix A.

A_n — area of reinforcement in bracket or corbel resisting tensile force N_{uc}, mm^2. Sub-section 11

A_l — total area of longitudinal reinforcement to resist torsion, mm^2. Sub-section 11

A_{ps} — area of pre-stressed reinforcement in tension zone, mm^2. Sub-sections 11 and 18

A'_s — area of non-pre-stressed tension reinforcement, mm^2. Sub-sections 8, 9, 10, 11, 12, and 18

A_{sh} — total cross-sectional area of transverse reinforcement (including cross-ties) within spacing s and perpendicular to dimension h_c. Appendix A

A_{st} — total area of longitudinal reinforcement, (bars or steel shapes), mm^2. Sub-section 10

A_t — area of structural steel shape, pipe, or tubing in a composite section, mm^2. Sub-section 10

A_t	area of one leg of closed stirrup resisting torsion within a distance s, mm^2. Sub-section 11
A_v	area of shear reinforcement within a distance s, or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s for deep flexural members, mm^2. Sub-sections 11, 12, and Appendix B.
A_v	total cross-sectional area of shear reinforcement within spacing, s, and perpendicular to longitudinal axis of structural member, mm^2. Appendix A
A_{vf}	area of shear-friction reinforcement, mm^2. Sub-section 11
A_{vh}	area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, mm^2. Sub-section 11
A_w	area of an individual wire to be developed or spliced, mm^2. Sub-section 12
A_1	loaded area. Sub-section 10 and Appendix B
A_2	maximum area of the portion of the supporting surface that is geometrically similar to and centric with the loaded area. Sub-section 10 and Appendix B
b	width of compression face of member, mm. Sub-sections 8,9,10,11,18 and Appendix B
b	effective compressive flange width of a structural member, mm. Appendix A
b	effective width of the section. Sub-section 9
b_o	perimeter of critical section for slabs and footings, mm. Sub-section 11 and Appendix B
b_t	width of that part of cross-section containing the closed stirrups resisting torsion. Sub-section 11
b_v	width of cross-section at contact surface being investigated for horizontal shear. Sub-section 17
b_w	web width, or diameter of circular section, mm. Sub-sections 11, 12, and Appendix B

c	distance from extreme compression fibre to neutral axis, mm. **Sub-section 10**
c_1	size of rectangular or equivalent rectangular column, captial, or bracket measured in the direction of the span for which moments are being determined, mm. **Sub-sections 11 and 13**
c_2	size of rectangular or equivalent rectangular column, captial, or bracket measured transverse to the direction of the span for which moments are being detemined, mm. **Sub-sections 11 and 13**
C	cross-section constant to define torsional properties. See Eq. (13-7) **Sub-section 13**
C_m	a factor relating actual moment diagram to an equivalent uniform moment diagram. **Sub-section 10**
C_t	factor relating shear and torsional stress properties. **Sub-section 11** $\dfrac{b_w d}{\sum x^2 y}$
d	distance from extreme compression fibre to centroid of tension reinforcement, mm. **Sub-section 7, 8, 10, 12, 13 and Appendix B**
d	distance from extreme compression fiber to centroid of non-pre-stressed tension reinforcement, mm. **Sub-section 18.**
d	distance from extreme compression fibre to centroid of longitudinal tension reinforcement, but need not be less than 0.80h for pre-stressed members, mm. (For circular sections, d need not be less than the distance from extreme compression fibre to centroid of tension reinforcement in opposite half of member). **Sub-section 11**
d	distance from extreme compression fibre to centroid of tension reinforcement for entire composite section, mm. **Sub-section 17**
d	effective depth of section. **Appendix A**
d'	distance from extreme compression fibre to centroid of compression reinforcement, mm. **Sub-sections 9 and 18**

d_b	nominal diameter of bar, wire, or pre-stressing strand, mm. Sub-sections 7 and 12
d_b	nominal diameter of bar, mm. Sub-section 3 and Appendix A
d_c	thickness of concrete cover measured from extreme tension fibre to centre of bar or wire located closest thereto, mm. Sub-section 10
d_c	thickness of cover to the centre of the first layer of bar in mm. Sub-section 9
d_p	diameter of pile at footing base. Sub-section 15
d_p	distance from extreme compression fibre to centroid of pre-stressed reinforcement. Sub-section 18
d_s	distance from extreme tension fibre to centroid of tension reinforcement, mm. Sub-section 9
D	dead loads, or related internal moments and forces. Sub-sections 9, 18, and 20
e	base of Napierian logarithms. Sub-section 18
E	load effects of earthquake, or related internal moments and forces. Sub-section 9 and Appendix A
E_c	modulus of elasticity of concrete, MPa. See Article 8.5.1. Sub-sections 8, 9, 10, and 19 and Appendix B
E_c	modulus of elasticity of concrete. Sub-section 9
E_{cb}	modulus of elasticity of beam concrete. Sub-section 13
E_{cc}	modulus of elasticity of column concrete. Sub-section 13
E_{cs}	modulus of elasticity of slab concrete. Sub-section 13
EI	flexural stiffness of compression member. See Eq. (10-10) and (10-11). Sub-section 10
E_s	modulus of elasticity of reinforcement, MPa. See Article 8.5.2 or 8.5.3. Sub-sections 8, 10 and Appendix B
E_s	Modulus of elasticity of steel. Sub-section 9
f'_c	specified compressive strength of concrete, MPa. Sub-sections 4, 8, 9, 10, 11, 12, 14, 18, 19 and Appendixes A and B

f'_{cr} required average compressive strength of concrete used as the basis for selection of concrete proportions, MPa. Sub-section 4

$\sqrt{f'_c}$ square root of specified compressive strength of concrete, MPa. Sub-sections 9, 11, 12, 15, 18, 19 and Appendix B

f'_{ci} compressive strength of concrete at time of initial pre-stress, MPa. Sub-section 18

$\sqrt{f'_{ci}}$ square root of compressive strength of concrete at time of initial pre-stress, MPa. Sub-section 18

f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa. Sub-sections 4, 9, 11, 12, and Appendix B

f_d stress due to unfactored dead load, at extreme fibre of section where tensile stress is caused by externally applied loads, MPa. Sub-section 11

f_{pc} compressive stress in concrete (after allowance for all pre-stress losses) at centroid of cross-section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, MPa. (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both pre-stress and moments resisted by pre-cast member acting alone). Sub-section 11

f_{pc} average compressive stress in concrete due to effective pre-stress force only (after allowance for all pre-stress losses,) MPa. Sub-section 18

f_{pe} compressive stress in concrete due to effective pre-stress forces only (after allowance for all pre-stress losses) at extreme fibre of section where tensile stress is caused by externally applied loads, MPa. Sub-section 11

f_{ps} stress in pre-stressed reinforcement at nominal strength. See text for units. Sub-sections 12 and 18

f_{pu} specified tensile strength of pre-stressing tendons, MPa. Sub-sections 11 and 18

f_{py} specified yield strength of pre-stressing tendons, MPa. Sub-section 18

f_r modulus of rupture of concrete, MPa. Sub-sections 9 and 18

f_s calcualted stress in reinforcement at service loads, MPa. Sub-section 10

f_s permissible tensile stress in reinforcement, MPa. Appendix B

f_s service stress. Sub-section 9

f_{se} effective stress in pre-stressed reinforcement (after allowance for all pre-stress losses). See text for units. Sub-sections 12 and 18

f_y specified yield strength of non-pre-stressed renforcement, MPa. Sub-sections 3, 7, 8, 9, 10, 11, 12, 18, 19, and Appendixes A and B

f_{yh} specified yeild strength of transverse reinforcement, MPa. Appendix A

F loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces. Sub-section 9

h overall thickness of member, mm. Sub-sections 9, 10, 11, 12, 13, 14, 18, 20 and Appendix A

h thickness of shell or folded plate, mm. Sub-section 19

h_c cross-sectional dimension of column core measured centre-to-centre of confining reinforcement Appendix A

h_v total depth of shearhead cross-section, mm. Sub-section 11

h_w total height of wall from base to top, mm. Sub-section 11

h_w height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered. Appendix A

H	loads due to height and pressure of soil, water in soil, or to other materials, or related internal moments and forces. Sub-section 9
I	moment of inertia of section resisting externally applied factored loads. Sub-section 11
I_b	moment of inertia about centroidal axis of gross section of beam as defined in Section 13.2.4 Sub-section 13
I_c	moment of inertia of gross section of column. Sub-section 13
I_{cr}	moment of inertia of cracked section transferred to concrete. Sub-section 9
I_e	effective moment of inertia for computation of deflection. Sub-section 9.
I_g	moment of inertia of gross concrete section about centoridal axis, neglecting reinforcement. Sub-sections 9 and 10
I_s	moment of inertia about centoridal axis of gross section of slab
	$h^3/12$ times width of slab defined in notations and Sub-section 13
I_{se}	moment of inertia of reinforcement about centroidal axis of member cross-section. Sub-section 10
I_t	moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross-section. Sub-section 10
k	effective length factor. Sub-section 14
k	wobble friction coefficient per meter of pre-stressing tendon. Sub-section 18
K	flexural stiffness of beam; moment per unit rotation. Sub-section 13
K	constant depending on shape of bending moment diagram. Sub-section 9
K_b	flexural stiffness of beam; moment per unit rotation. Sub-section 13
K_c	flexural stiffness of column; moment per unit rotation. Sub-section 13

K_s	flexural stiffness of slab; moment per unit rotation. Sub-section 13
K_t	torsional stiffness of torsional member; moment per unit rotation. Sub-section 13
l	span length of beam or one-way slab, as defined in section 8.7; clear projection of cantilever, mm. Sub-section 9
l	length of span of two-way flat plates in direction parallel to that of the reinforcement being determined, mm. See Eq. (18-8). Sub-section 18
l_a	additional embedment length at support or at point of inflection, mm. Sub-section 12
l_c	vertical distance between supports, mm. Sub-section 14
l_d	development length, mm. Sub-sections 7, 12 and Appendix A
l_{dh}	development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook (point of tangency) plus redius of bend and one bar diameter), mm l_{hb} x applicable modification factors. Sub-section 12
l_{dh}	development length for a bar with a standard hook as defined in Eq. (A-5). Appendix A
l_{hb}	basic development length of standard hook in tension, mm. Sub-section 12
l_n	clear span for positive moment or shear and average of adjacent clear spans for negative moment. Sub-section 8
l_n	clear span measured face-to-face of supports Sub-section 11
l_n	length of clear span in long direction of two-way construction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases. Sub-section 9

l_n length of clear span in direction that moments are being determined, measured face-to-face of supports. Sub-section 13

l_o minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, mm. Appendix A

l_t span of member under load test (shorter span of flat slabs and of slabs supported on four sides). Span of member, except as provided in Article 20.4.9, is distance between centres of supports or clear distance between supports plus depth of member, whichever is smaller, mm. Sub-section 20

l_u unsupported length of compression member. Sub-section 10

l_v length shearhead arm from centroid of concentrated load or reaction, mm. Sub-section 11

l_w horizontal length of wall, mm. Sub-section 11

l_w length of entire wall (diaphragm) or a segment of wall (diaphragm) considered in direction of shear force. Appendix A

l_x length of pre-stressing tendon element from jacking end to any point x, m. See Eq. (18-1) and (18-2). Sub-section 18

l_1 length of span in direction that moments are being determined, measured center-to-center of supports. Sub-section 13

l_2 length of span transverse to l_1, measured centre-to-centre of supports. See also Articles 13.6.2.3 and 13.6.2.4. Sub-section 13

L live loads, or related internal moments and forces. Sub-sections 9, 18 and 20

M design moment. Appendix B

M_a maximum moment in member at stage deflection is computed. Sub-section 9

M_c factored moment to be used for design of compression member. Sub-section 10

M_{cr}	cracking moment. See Article 9.5.2.3. Sub-section 9
M_{cr}	moment causing flexural cracking at section due to externally applied loads. See Article 11.4.2.1. Sub-section 11
M_m	modified moment. Sub-section 11
M_{max}	maximum factored moment at section due to externally applied loads. Sub-section 11
M_o	total factored static moment. Sub-section 13
M_n	nominal moment strength at section, Nm. Sub-section 12 $A_s f_y (d - a/2)$
M_p	required plastic moment strength of shearhead cross-section. Sub-section 11
M_s	portion of slab moment balanced by support moment. Appendix A
M_u	factored moment at section. Sub-section 11
M_v	moment resistance contributed by shearhead reinforcement. Sub-section 11
M_{1b}	value of smaller factored end moment on compression member due to the loads that result in no appreciable sidesway, calculated by conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature. Sub-section 10
M_{2b}	value of larger factored end moment on compression member due to loads that result in appreciable sidesway calculated by conventional elastic frame analysis. Sub-section 10
M_{2s}	value of larger factored end moment on compression member due to loads that result in appreciable sidesway calculated by conventional elastic frame analysis. Chapter 10
n	modular ratio of elasticity. Appendix B. E_s/E_c
N	design axial load normal to cross-section occurring simultaneously with V; to be taken as positive for compression, negative for tension, and to include effects of tension due to creep and shrinkage. Appendix B

N_c tensile force in concrete due to unfactored dead load plus live load (D + L). **Sub-section 18**

N_u factored axial load normal to cross-section occurring simultaneously with V_u; to be taken as positive for compression, negative for tension,a dn to include effects of tension due to creep and shrinkage. **Sub-section 11**

N_{uc} factored tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension. **Sub-section 11**

P_b nominal axial load strength at balanced strain conditions. See Article 10.3.2. **Sub-sections 9 and 10**

P_c critical load. See Eq. (10-9). **Sub-section 10**

P_n nominal axial load strength at given eccentricity. **Sub-sections 9 and 10**

P_o nominal axial load strength at zero eccentricity. **Sub-section 10**

P_o coefficient which depends on the percentages of tension and compression steel. **Sub-section 9**

P_s pre-stressing tendon force at jacking end. **Sub-section 18**

P_u factored axial load at given eccentricity $\leq \phi P_n$. **Sub-sections 9 and 10**

P_{nw} nominal axial load strength of wall designed by Article 14.4. **Sub-section 14**

P_x pre-stressing tendon force at any point x. **Sub-section 18**

r radius of gyration of cross-section of a compression member. **Sub-section 10**

r_b^{-1} Curvature at midspan, for cantilevers at the support return. **Sub-section 9**

r_{cs}^{-1} Shrinkage curvature. **Sub-section 9**

r_x^{-1} Curvature at x. **Sub-section 9**

r_z^{-1} curvature at z. **Sub-section 9**

s standard deviation, MPa. **Sub-section 4**

s	spacing of shear or torsion reinforcement in direction parallel to longitudinal reiforcement, mm. Sub-section 11
s	spacing of stirrups or ties, mm. Sub-section 12
s	spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm. Appendix A
s	spacing of shear reinforcement in direction parallel to longitudinal reinforcement, mm. Appendix B
s_o	maximum spacing of transverse reinforcement, mm. Appendix A
s_w	spacing of wire to be developed or spliced, mm. Sub-section 12
s_1	spacing of vertical reinforcement in wall, mm. Sub-section 11
s_2	spacing of shear or torsion reinforcement in direction perpendicular to longitudinal reinforcement - or spacing of horizontal reinforcement in wall, mm. Sub-section 11
T	cumulative effects of temperatures, creep, shrinkage, and differential settlement. Sub-section 9
T_c	nominal torsional moment strength provided by concrete. Sub-section 11
T_n	nominal torsional moment strength. Sub-section 11
T_s	nominal torsional moment strength provided by torsion reinforcement. See Article 11.6.8.3. Sub-section 11
T_u	factored torsional moment at section. Sub-section 11
U	required strength to resist factored loads or related internal moments and forces. Sub-section 9
v	redesign shear stress. Appendix B
v_c	permissible shear stress carried by concrete, MPa. Sub-section 11 and Appendix B

v_h	permissible horizontal shear stress, MPa. Appendix B
V	design shear force at section. Appendix B
V_c	nominal shear strength provided by concrete. Sub-sections 8, 11 and Appendix A
V_{ci}	nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment. Sub-section 11
V_{cw}	nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web. Sub-section 11
V_d	shear force at section due to unfactored dead load. Sub-section 11
V_e	design shear force determined from Article A.7.1.1 or A.7.1.2. Appendix A
V_i	factored shear force at section due to externally applied loads occurring simultaneously with M_{max}. Sub-section 11
V_n	nominal shear strength. Sub-section 11 and Appendix A
V_{nh}	nominal horizontal shear strength. Sub-section 17
V_p	vertical component of effective pre-stress force at section. Sub-section 11
V_s	nominal shear strength provided by shear reinforcement. Sub-section 11
V_u	factored shear force at section. Sub-sections 11, 12, and 17, and Appendix A
w_c	mass of concrete, kg/m^3. Sub-sections 8 and 9
w_d	factored dead load per unit area. Sub-section 13
w_t	factored live load per unit area. Sub-section 13
w_u	factored load per unit length of beam or per unit area of slab. Sub-section 8
w_u	factored load per unit area. Sub-section 13
W	wind load, or related internal moments and forces. Sub-section 9

x shorter overall dimension of rectangular part of cross-section. Sub-sections 11 and 13

x_1 shorter centre-to-centre dimension of closed rectangular stirrup. Sub-section 11

y longer overall dimension of rectangular part of cross-section. Sub-sections 11 and 13

y_t distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension. Sub-sections 9 and 11

y_1 longer centre-to-centre dimension of closed rectangular stirrup. Sub-section 11

z quantity limiting distribution of flexural reinforcement. See Article 10.6 Sub-section 10.

α (alpha) angle between inclined stirrups and longitudinal axis of member. Sub-section 11 and Appendix B.

α total angular change of pre-stressing tendon profile in radians from tendon jacking end to any point x. Sub-section 18

α ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam. Sub-sections 9 and 13

$$\frac{E_{cb}I_b}{E_{cs}I_s}$$

α_c ratio of flexural stiffness f columns above and below the slab to combined flexural stiffness of the slabs and beams at a joint taken in the direction of the span for which moments are being determined. Sub-section 13.

$$\frac{\sum K_c}{\sum(K_s + K_b)}$$

α_c coefficient defining the relative contribution of concrete strength to wall strength. See Eq. (A-7) Appendix A

α_f angle between shear-friction reinforcement and shear plane. Sub-section 11

α_m average value of α for all beams on edges of a panel. Sub-section 9

α_{min}	minimum α_s to satisfy Article 13.6.10(a). Sub-section 13
α_t	coefficient as a function of y_1/x_1. See Article 11.6.10.1 Sub-section 11
α_v	ratio of stiffness of shearhead arm to surrounding composite slab section. See Article 11.11.4.5. Sub-section 11
α_1	in direction of l_1. Sub-section 13
α_2	in direction of l_2. Sub-section 13
β (beta)	ratio of clear spans in long to short direction of two-way slabs. Sub-section 9
β	ratio of long side to short side of footing. Sub-section 15
β	depth factor
β_a	ratio of dead load per unit area to live load per unit area (in each case without load factors). Sub-section 13
β_b	ratio of area of reinforcement cut off to total area of tension reinforcement at section. Sub-section 12
β_c	ratio of long side to short side of concentrated load or reaction area. Sub-section 11 and Appendix B
β_d	absolute value of ratio of maximum factored dead load moment to maximum factored total load moment, always positive. Sub-section 10
β_s	ratio of length of continuous edges total perimeter of a slab panel. Sub-section 9
β_t	ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, centre-to-centre of supports. Sub-section 13 $\frac{E_{cb}C}{2E_{cs}I_s}$
β_1	factor defined in Articles 10.2.7.3. Sub-sections 8 and 10
β_1	factor defined in Article 10.2.7.1 Sub-section 18

γ_f (gamma)	fraction of unbalanced moment transferred by flexure at slab-column connections. See Article 13.3.3.2 Sub-sections 11 and 13
γ_p	factor for type of pre-stressing tendon. Sub-section 18 0.40 for f_{py}/f_{pu} not less than 0.85 0.28 for f_{py}/f_{pu} not less than 0.90
γ_v	fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections. See Article 11.12.2.3 Sub-section 11 $1 - \gamma_f$
δ_b (delta)	moment maginification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member. Sub-section 10
δ_s	moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads. Sub-section 10
δ_s	factor defined by Eq. (13-5). See Article 13.6.10. Sub-section 13
δ_{cs}	free shrinkage strain. Sub-section 9
η (eta)	number of identical arms of shearhead. Sub-section 11
λ (lambda)	multiplier for additional long-time deflection as defined in Article 9.5.2.5. Sub-section 9
λ	correction factor related to unit mass of concrete. Sub-section 11
μ (mu)	coefficient of friction. See Article 11.7.4.3 Sub-section 11
μ	curvature friction coefficient. Sub-section 18
ξ (xi)	time-dependent factor for sustained load. See Article 9.5.2.5 Sub-section 9
ρ (rho)	ratio of non-pre-stressed tension reinforcement. Sub-sections 8, 10, 11, 18 and Appendixes A and B A_s/bd
ρ'	ratio of non-pre-stressed compression reinforcement. Sub-section 8 A'_s/bd

ρ' reinforcement ratio for non-pre-stressed compression reinforcement, A'_s/bd. Sub-section 9

ρ' ratio of compression reinforcement. Sub-section 18 A'_s/bd

ρ_b reinforcement ratio producing balanced strain conditions. See Article 10.3.2 Sub-sections 8 and 10

ρ_g ratio of total reinforcement area to cross-sectional area of column. Appendix A

ρ_h ratio of horizontal shear reinforcement area to gross concrete area of vertical section. Sub-section 11

ρ_n ratio of vertical shear reinforcement area to gross concrete area of horizontal section. Sub-section 11

ρ_n ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv}. Appendix A

ρ_p ratio of pre-stressed reinforcement. Sub-section 18 A_{ps}/bd

ρ_s ratio of volume of spiral reinforcement to total volume of core (out-to-out spirals) of a spirally reinforced compression member. Sub-section 10

ρ_s ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out) Appendix A

ρ_v A_{sv}/A_{cv}; where A_{sv} is the projection on A_{cv} of area of distributed shear reinforcement crossing the plane of A_{cv}. Appendix A

ρ_w $A_s/b_w d$ Sub-section 11

ϕ (phi) strength reduction factor. See Section 9.3. Sub-sections 8, 9, 10, 11, 14, 15, 17, 18, 19 and Appendix A.

ϕ strength reduction factor. See Article B.2.1 Appendix B

creep coefficient

ω (omega) pf_y/f'_c Sub-section 18

ω' $p'f_y/f'_c$ Sub-section 18

ω_p $P_p f_{ps}/f'_c$ Sub-section 18

$\omega_w \; \omega_{pw} \; \omega'_w$ reinforcement indices for flanged sections computed as for ω, ω_p, and ω' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only. **Sub-section 18**

$\sum x^2 y$ torsional section properties. See Article 11.6.1.1 and 11.6.1.2. **Sub-section 11**

APPENDIX D

METAL REINFORCEMENT INFORMATION

APPENDIX D - METAL REINFORCEMENT INFORMATION

As an aid to users of the Building Code, information on sizes, areas, and weights of various metal reinforcement is presented.

ASTM STANDARD REINFORCING BARS

Bar size	Nominal diameter, mm	Nominal area, mm^2	Nominal mass, kg/m
#10	11.3	100	0.785
15	16.0	200	1.570
20	19.5	300	2.355
25	25.2	500	3.925
30	29.9	700	5.495
35	35.7	1000	7.850
45	43.7	1500	11.775
55	56.4	2500	19.625

ASTM STANDARD PRESTRESSING STRANDS

Type*	Nominal diameter, mm	Nominal area, mm^2	Nominal mass, kg/m
Seven-wire strand (Grade 250)	6.35	23.22	0.182
	7.94	37.42	0.294
	9.53	51.61	0.405
	11.11	69.68	0.548
	12.70	92.90	0.730
	15.24	139.35	1.094
Seven-wire strand (Grade 270)	9.53	54.84	0.432
	11.11	74.19	0.582
	12.70	98.71	0.775
	15.24	140.00	1.102
Prestressing wire	4.88	18.7	0.146
	4.98	19.4	0.149
	6.35	32	0.253
	7.01	39	0.298
Prestressing bars (smooth)	19	284	2.23
	22	387	3.04
	25	503	3.97
	29	639	5.03
	32	794	6.21
	35	955	7.52
Prestressing bars (deformed)	15	181	1.46
	20	271	2.22
	26	548	4.48
	32	806	6.54
	36	1019	8.28

*Availability of some tendon sizes should be investigated in advance.

ASTM STANDARD WIRE REINFORCEMENT

W&D size		Nominal diameter, mm	Nominal area, mm^2	Nominal mass, kg/m
Smooth	Deformed			
W31	D31	15.95	200	1.569
W30	D30	15.70	194	1.518
W28	D28	15.16	181	1.417
W26	D26	14.60	168	1.390
W24	D24	14.05	155	1.214
W22	D22	13.44	142	1.113
W20	D20	12.80	129	1.012
W18	D18	12.14	116	0.911
W16	D16	11.46	103	0.810
W14	D14	10.72	90	0.708
W12	D12	9.91	77	0.607
	D11	9.50	71	0.557
W10	D10	9.04	64	0.506
	D9	8.58	58	0.455
W8	D8	8.10	52	0.405
W7	D7	7.57	45	0.354
W6	D6	7.01	39	0.304
W5.5		6.71	36	0.278
W5	D5	6.40	32	0.253
W4.5		6.10	29	0.228
W4	D4	5.72	26	0.202
W3.5		5.36	23	0.177
W3		4.95	19.4	0.152
W2.5		4.52	16.1	0.127
W2		4.04	12.9	0.101

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Standard Building Code

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APPENDIX E

FIRE-RESISTANCE

APPENDIX E: FIRE-RESISTANCE

E.0 General

A concrete element of construction when subjected to fire undergoes a gradual reduction in strength and ridgidity. There are three conditions to be considered:

retention of structural strength
resistance to penetration of flames
resistance to heat transmission

The first criterion is applicable to all elements of construction, while walls and floors which perform a separating function are also judged on the other two criteria.

Factors that influence the fire-resistance of concrete structures are given below. Some of these cannot at this time be taken into account quantitatively.

Size and shape of the element
Type of concrete
Type of reinforcement
Protective concrete cover provided to reinforcement or tendons
The load supported
The conditions of restraint

Concretes made with siliceous aggregates have a tendency to spall when exposed to high temperatures but this can be reduced by the incorporation of supplementary reinforcement in the concrete cover. Spalling does not generally occur with either calcareous or lightweight aggregates. Concretes made with lightweight aggregate possess a superior insulation in comparison with siliceous and calcareous aggregate concretes.

Concrete pre-stressing tendons and reinforcement show a reduction in strength at high temperatures. Evidence from investigation has shown that in the region of 400°C tendons are likely to retain about 50% of their strength at ambient tempertures and that for reinforcement a similar reduction in strength occurs at about 550°C.

The fire-resistance of structural elements is generally determined when the specimen is supporting its service load, taken as the sum of all the characteristic dead and imposed loads. The tables which follow show the minimum sizes for various elements when supporting these loads; any reduction in load would be reflected by an increase in fire-resistance but sufficient data are not available to define the relationship.

E.1 Beams

The fire-resistance of a reinforced or pre-stressed concrete beam depends on the amount of protective cover, consisting of concrete with or without an insulating encasement, provided to the reinforcement of tendons. It is also necessary for the beam to have a minimum width to avoid failure of the concrete before the reinforcement or tendons reach the critical temperature.

Typical performances are given in Table E-1 for reinforced concrete beams and in Table E-2 for pre-stressed concrete beams, both for siliceous aggregate concrete and for lightweight aggregate concrete.

TABLE E-1
FIRE-RESISTANCE OF REINFORCED CONCRETE BEAMS

Description	Dimension of concrete to give a fire resistance in hours					
	4	3	2	1.5	1	0.5
	mm	mm	mm	mm	mm	mm
(1) Siliceous aggregate concrete:						
a. average concrete cover to main reinforcement	65*	55*	45*	35	25	15
b. beam width	280	240	180	140	110	80
(2) As (1) with cement or qupsum plaster 15 mm thick on light mesh reinforcement:						
a. average concrete cover to main reinforcement	50*	40	30	20	15	15
b. beam width	250	210	170	110	85	70
(3) As(1) with vermiculite/gypsum plaster+ or sprayed asbestos** 15mm thick:						
a. average concrete cover to main reinforcement	25	15	15	15	15	15
b. beam width	170	145	125	85	60	60
(4) Lightweight aggregate concrete:						
a. average concrete cover to main reinforcement	50	45	35	30	20	15
b. beam width	250	200	160	130	100	80

*Supplementary reinforcement, to hold the concrete cover in position, may be necessary.
+ Vermiculite/gypsum plaster should have a mix ratio in the range of 1.5: 1 by volume.
** Sprayed asbestos should conform to BS 3590.

For I-section beams the average concrete cover to the reinforcement in the lower flange should be increased in the ratio

$$\frac{b}{b_w} \text{ for } \frac{b}{3} \leq b_w < b$$

where b is the breadth of lower flange,

b_w is the thickness of web

Where $b_w < b/3$ the tabulated data do not apply and additional protection may be required for the web and flange. Where large scale production of a section is intended, a fire test is highly desirable in the interests of economy as well as safety.

TABLE E-2
FIRE-RESISTANCE OF PRE-STRESSED CONCRETE BEAMS

Description	Dimension of concrete to give a fire resistance in hours					
	4	3	2	1.5	1	0.5
	mm	mm	mm	mm	mm	mm
(1) Siliceous aggregate concrete:						
A. average concrete cover to tendons	100*	85*	65*	50*	40	25
b. beam width	280	240	180	140	110	80
(2) As (1) with vermiculite concrete slabs' 15 mm thick, used as permanent shuttering:						
a. average concrete cover to tendons	75*	60	45	35	25	15
b. beam width	210	170	125	100	70	70
(3) As (2) but with 25 mm thick slabs						
a. average concrete cover to tendons	65	50	35	25	15	15
b. beam width	180	140	100	70	60	60
(4) As (1) with 15 mm thick gypsum plaster with light mesh reinforcement:						
a. average concrete cover to tendons	90*	75	50	40	30	15
b. beam width	250	210	170	110	85	70
(5) As (1) with vermiculite/gypsum plaster+, or sprayed asbestos**, 15 mm thick:						
a. average concrete cover to tendons	75*	60	45	30	25	15
b. beam width	170	145	125	85	60	60
(6) As (5) but with 25 mm thick coating:						
a. average concrete cover to tendons	50	45	30	25	15	15
b. beam width	140	125	85	70	60	60
(7) Lightweight aggregate concrete:						
a. average concrete cover to tendons	80	65	50	40	30	20
b. beam width	250	200	160	130	100	80

*Supplementary reinforcement, to hold the concrete cover in position, may be necessary.

+ Vermiculite/gypsum plaster should have a mix ratio in the range of 1.5: 1 by volume.

** Sprayed asbestos should conform to BS 3590.

E.2 Floors

The fire-resistance of a floor is dependent upon the minimum thickness of the concrete section and the average concrete cover to the reinforcement in the tensile zone. The performance of some typical reinforced concrete and pre-stressed concrete floors is given below.

In estimating the thickness of concrete, non-combustible screeds or floor finishes may be taken into account.

The average concrete cover is determined by summing the product of the cross-sectional area of each bar or tendon and the distance from the surface of the bar to the nearest relevant exposed face and dividing it by the total area of these bars or tendons. Only those bars or tendons provided for the purpose of resisting tension due to ultimate loads should be considered in this calculation.

The tables give the average concrete cover required to provide the stated fire-resistance but in no case should the nominal concrete cover to any bar or tendon be less than half this value nor less than the value given for the half hour period appropriate to that form of construction.

In addition, for certain types of floor made with siliceous aggregate concrete it will be necessary to consider the provision of supplementary reinforcement to hold the concrete cover in position.

Supplementary reinforcement will be requried in cases indicated in the table when no ceiling protection is provided (E.2.1) and the cover to all bars and tendons is more than 40 mm.

When used, supplementary reinforcement should consist of expanded metal lath or a wire fabric not lighter than 0.5 kg/m^2 (2 mm diameter wires at not more than 100 mm centres) or a continuous arrangmeent of links at not more than 200 mm centres incorporated in the concrete cover at a distance not exceeding 20 mm from the face.

In the absence of adequate test data on lightweight concrete floors, these should be treated as dense concrete floors even though their fire-resistance might be expected to be somewhat superior.

Table E-3 Fire resistance of reinforced concrete floors (siliceous or calcareous aggregate)

Floor construction		Minimum dimension to give fire resistance in hours					
		4	3	2	1½	1	½
		mm	mm	mm	mm	mm	mm
(1) Solid slab	Average cover to reinforcement	25	25	20	20	15	15
	Depth, overall†	150	150	125	125	100	100
(2) Cored slabs in which the cores are circular or are higher than wide. Not less than 50 % of the gross cross section of the floor should be solid material	Average cover to reinforcement	25	25	20	20	15	15
	Thickness under cores	50	40	40	30	25	20
	Depth, overall†	190	175	160	140	110	100
(3) Hollow box section with one or more longitudinal cavities which are wider than high	Average cover to reinforcement	25	25	20	20	15	15
	Thickness of bottom flange	50	40	40	30	25	20
	Depth, overall†	230	205	180	155	130	105
(4) Ribbed floor with hollow infill blocks of clay, or inverted T-section beams with hollow infill blocks of concrete or clay. A floor in which less than 50 % of the gross cross section is solid material must be provided with a 15 mm plaster coating on soffit	Average cover to reinforcement	25	25	20	20	15	15
	Width of rib, or beam, at soffit	125	100	90	80	70	50
	Depth, overall†	190	175	160	140	110	100
(5) Upright T-sections	Average bottom cover to reinforcement	65*	55*	45*	35	25	15
	Side cover to reinforcement	65	55	45	35	25	15
	Least width of downstanding leg	150	140	115	90	75	60
	Thickness of flange†	150	150	125	125	100	90
(6) Inverted channel sections with radius at intersection of soffits with top of leg not exceeding depth of section	Average bottom cover to reinforcement	65*	55*	45*	35	25	15
	Side cover to reinforcement	40	30	25	20	15	10
	Least width of each downstanding leg	75	70	60	45	40	30
	Thickness at crown†	150	150	125	125	100	90
(7) Inverted channel sections or U-sections with radius at intersection of soffits with top of leg exceeding depth of section	Average bottom cover to reinforcement	65*	55*	45*	35	25	15
	Side cover to reinforcement	40	30	25	20	15	10
	Least width of each downstanding leg	70	60	50	40	35	25
	Thickness at crown†	150	150	100	100	75	65

* Supplementary reinforcement, to hold the concrete cover in position, may be necessary.
† Non-combustible screeds and finishes may be included in these dimensions.

Table E-4 Fire resistance of prestressed concrete floors (siliceous or calcareous aggregate)

Floor construction		Minimum dimension to give fire resistance in hours					
		4	3	2	1½	1	½
		mm	mm	mm	mm	mm	mm
(1) Solid slabs	Average cover to tendons	65*	50*	40	30	25	15
	Depth, overall†	150	150	125	125	100	90
(2) Cored slabs in which the cores are circular or are higher than wide. Not less than 50 % of the gross cross section of the floor should be solid material	Average cover to tendons	65*	50*	40	30	25	15
	Thickness under cores	50	40	40	30	25	20
	Depth, overall†	190	175	160	140	110	100
(3) Hollow box section with one or more longitudinal cavities which are wider than high	Average cover to tendons	65*	50*	40	30	25	15
	Thickness of bottom flange	65	50	40	30	25	25
	Depth, overall†	230	205	180	155	130	105
(4) Ribbed floor with hollow infill blocks of clay, or inverted T-sections with hollow infill blocks of concrete or clay. A floor in which less than 50 % of the gross cross section is solid material must be provided with a 15 mm plaster coating on soffit	Average cover to tendons	65*	50*	40	30	25	15
	Width or rib, or beam, at soffit	125	100	90	80	70	50
	Depth, overall†	190	175	160	140	110	100
(5) Upright T-sections	Average bottom cover to tendons	100*	85*	65*	50*	40	25
	Side cover to tendons	100	85	65	50	40	25
	Least width of downstanding leg	250	200	150	110	90	60
	Thickness of flange†	150	150	125	125	100	90
(6) Inverted channel section with radius at intersection of soffits with top of leg not exceeding depth of section	Average bottom cover to tendons	100*	85*	65*	50*	40	25
	Side cover to tendons	50	45	35	25	20	15
	Least width of each downstanding leg	125	100	75	55	45	30
	Thickness at crown†	150	150	125	125	100	90
(7) Inverted channel or U-section with radius at intersection of soffits with top of leg exceeding depth of section	Average bottom cover to tendons	100*	85*	65*	50*	40	25
	Side cover to tendons	50	45	35	25	20	15
	Least width of each downstanding leg	110	90	70	50	45	30
	Thickness at crown†	150	150	125	125	100	90

* Supplementary reinforcement, to hold the concrete cover in position, may be necessary.
† Non-combustible screeds and finishes may be included in these dimensions.

E.2.1 ADDITIONAL PROTECTION TO FLOORS

The fire-resistance of any given form of floor construction may be improved by the provisions of an insulating finish on the soffit or by a suitable suspended ceiling, some examples of which are given in Table E-5.

TABLE E-5
EFFECT OF CEILING TREATMENT

Ceiling finish	Thickness of finish to give an increase in fire resistance in hours 3	2	1.5	1	0.5
	mm	mm	mm	mm	mm
(1) Vermiculite/gypsum plaster* or sprayed asbestos+ applied to the soffit of floor Types 1,2 or 3	25	15	15	10	10
(2) Vermiculite/gypsum plaster* or sprayed asbestos+ on expanded metal as a suspended ceiling to floor Types 4 or 5	15	10	10	10	10
(3) Gypsum/sand or cement/sand on expanded metal as a suspended ceiling to any floor type	25	20	15	10	10

*Vermiculite/gypsum plaster should have a mix ratio in the range of 1.5-2: 1 by volume.
+Sprayed asbestos should conform to BS 3590.

E.3 Columns

The minimum dimension of a column is a determining factor in the fire-resistance it can provide. The dimensions given in the table below relate to columns which may be exposed to fire on all faces when subjected to service loads. The use of limestone or other calcareous aggregates will as indicated reduce spalling and allow a reduction in the size of the section. Where siliceous aggregates are used, the concrete cover to the main bars should not exceed 40 mm without the use of supplementary reinforcement.

Supplementary reinforcement should consist of either a wire fabric not lighter than 0.5 kg/m^2 (2 mm diameter wires at not more than 100 mm centres) or a continuous arrangement of links at not more than 200 mm centres incorporated in the concrete cover at a distance not exceeding 20 mm from the face.

When supplementary reinforcement is used under (2) in Table 6 to obtain a reduced size of column it should be placed at mid-cover but not more than 20 mm from the face and should be in the shape of a rectangular or circular cage.

TABLE E-6

FIRE-RESISTANCE OF CONCRETE COLUMNS (ALL FACES EXPOSED)

Type of construction	Dimension of concrete to give fire-resistance in hours					
	4	3	2	1.5	1	0.5
	mm	mm	mm	mm	mm	mm
(1) Siliceous aggregate concrete:						
a. without additional protection	450	400	300	250	200	150
b. with cement or gypsum plaster 15mm thick on light mesh reinforcement	300	275	225	150	150	150
c. with vermiculite/gypsum plaster* or sprayed asbestos** 15 mm thick	275	225	200	150	120	120
(2) Limestone aggregate concrete of siliceous aggregate concrete with supplementary reinforcement in concrete cover	300	275	225	200	190	150
(3) Lightweight aggregate concrete	300	275	225	200	150	150

* Vermiculite/gypsum plaster should have a mix ratio in the range of 1.5 - 2: 1 by volume.

** Sprayed asbestos should conform to BS 3590.

Columns which are built into fire-resistance walls to their full height are likely to be exposed to fire on one face only. Data given in the table below apply to the situation where the face of the column is flush with the wall or that part embedded in the wall is structurally adequate to support the load, provided that any opening in the wall is not nearer to the column than the minimum dimension specified in the table for that column.

TABLE E-7

FIRE-RESISTANCE OF CONCRETE COLUMNS (ONE FACE EXPOSED)

Type of construction	Dimension of concrete to give a fire-resistance in hours					
	4	3	2	1.5	1	0.5
	mm	mm	mm	mm	mm	mm
Siliceous aggregate concrete:						
a. without additional protection	180	150	100	100	75	75
b. with vermiculite/gypsum plaster* or sprayed asbestos** 15 mm thick on exposed faces	125	100	75	75	65	65

* Vermiculite/gypsum plaster should have a mix ratio in the range of 1.5 - 2 : 1 by volume.

** Sprayed asbestos should conform to BS 3590.

E.4 Walls

E.4.1 CONCRETE WALLS CONTAINING AT LEAST 1.0% OF VERTICAL REINFORCEMENT.

The fire-resistances of such walls are given in the table below. The minimum thicknesses shown are for siliceous aggregate concrete. When using lightweight aggregate concrete a reduction in thickness is possible when confirmed by a test. Concrete cover to the reinforcement should be not less than 15 mm for fire-resistance up to one hour and not less than 25 mm for higher periods. Walls containing vertical reinforcement less than 1.0% are considered as plain concrete for fire-resistance purposes unless shown otherwise by a test (see E.4.2).

TABLE E-8

FIRE-RESISTANCE OF SILICEOUS AGGREGATE CONCRETE WALLS CONTAINING AT LEAST 1.0% OF VERTICAL REINFORCEMENT AND EXPOSED TO FIRE ON ONE FACE ONLY

Type of construction	Dimension of concrete to give a fire-resistance in hours					
	4 mm	3 mm	2 mm	1.5 mm	1 mm	0.5 mm
None	180	150	100	100	75	75
Vermiculite/gypsum plaster* or sprayed asbestos** 15 mm thick on exposed faces	125	100	75	75	65	65

* Vermiculite/gypsum plaster should have a mix ratio in the range of 1.5 - 2 : 1 by volume.

** Sprayed asbestos should conform to BS CP 110-1972.

Walls exposed to fire on more than one face should be regarded as columns (see E.3).

E.4.2 PLAIN CONCRETE WALLS

From the limited data available, the fire-resistance of plain siliceous aggregate concrete walls can be taken as follows:

concrete 150 mm thick:	1 hour
concrete 175 mm thick:	1-1/2 hours

APPENDIX F

RECOMMENDATION

APPENDIX F

RECOMENDATIONS FOR THE USE OF STRUCTURAL REINFORCED CONCRETE PROVISIONS REGARDING THE USE OF AGGREGATES

F.0 Foreword

This Appendix provides basic guidelines on the use of aggregates for normal structural concrete (i.e. without admixtures) in the territories to which the Code is related. It is intended to ensure avoidance of major errors which have been identified by regional practice and a general knowledge of concrete technology. Cautions of interest to particular territories will also be briefly set forth.

It assumes that the design and construction of such concrete works has been directed by competent professional designers.

F.1 General Provisions

Aggregates shall in general satisfy ASTM C33 (to which the attention of readers is drawn) or BS882 or similar established standard specifications for aggregates, in particular as regards grading (particle-size distribution) and allowable limits of deleterious substances. They shall be hard, clean (that is, free from either excessive silt or clay), and shall be from sources proven to provide strong and durable concrete in the local context. New or unproven aggregates shall not be used in major works without substantiation of the fitness of these proposed aggregates for their intended purpose in the works. There shall not be an abundance of shaly or slatey (flat) and/or elongated particles in aggregates intended for reinforced or pre-stressed concrete, and the gradings shall in particular not exhibit substantial deviation from those in the standards referred to above, except justified by local performance to the satisfaction of the Building Official.

Aggregates, in particular coarse aggregates, shall be free from adherent coatings of mud, silt, or clay, and from contamination by organic materials, in particular decaying plant or animal matter; in the case of petroleum producing areas, they shall additionally be substantially free or oil, coal, peat and other similar hydrocarbon contamination.

F.1.1 CHLORIDES IN AGGREGATES

Aggregates, particularly fine aggregates, shall in general be from land and not from marine sources.

It is nonetheless recognized that aggregates from marine sources may continue to be used, particularly where their use is not prevented on environmental considerations (such as beach erosion prevention). In such cases, and particularly when fine aggregates are obtained from zones between high and low tides, they shall be washed to ensure that they enable the resulting concrete to satisfy the following provisions:

The total chloride content of the concrete mix arising from the aggregate, together with that from any admixtures and any other sources should not in any circumstances exceed the following limits:

TYPE OR USE OF CONCRETE	MAXIMUM TOTAL CHLORIDE CONTENT EXPRESSED AS PERCENTAGE OF CHLORIDE ION BY WEIGHT OF CEMENT
Pre-stressed concrete Structural concrete that is steam cured Concrete for any use made with sulphate-resisting Portland Cement complying with ASTM C150-81 Type V cement or BS 4027	0.06
Reinforced concrete (or concrete containing embedded metal) made with cement complying with ASTM C150-81 Type I or BS 12.	0.35 for 95% of test results with no result greater than 0.50.

Note: % chloride ion x 1.648 = % equivalent sodium chloride
% chloride ion x 1.565 = % equivalent calcium chloride

F.1.2 CEMENT

Cement shall be Ordinary Portland or Sulphate-Resisting Portland Cement (ASTM Type I and V or BS 12 and BS 4027: 1980 respectively) and shall be from a source from which cement of satisfactory local performance has been obtained in the past.

Whenever it is necessary to change the cement EITHER to a type not complying with the above specifications OR to cement from an unproven source, during the course of construction trial mixes shall be made to check whether, notwithstanding alleged compliance with the specifications, variations of practical construction significance are exhibited by the new cement. In particular, setting times and early rates of strength gain shall be checked, since these may affect, respectively, the timing of finishing operations and the stripping of formwork, as well as 7-day to 28-day strength ratios (see Clause F.1.7(b))

F.1.3 PROCESSES

In the manufacture of concrete: batching, mixing, transportation, placing, compaction, finishing, and curing shall be so executed as to ensure the achievement and maintenance of homogeneity of the fresh concrete, and a virtual absence of segregation.

F.1.4 COARSE AGGREGATE (ABOVE 5 MM)

Coarse aggregates for use in normal structural concrete (i.e. where no special requirements such as concrete for exposed architectural uses, marine works, water-retaining or water excluding works, concrete to be subjected to severe abrasion, concrete containing admixtures, etc.) shall not contain more than 5% by weight of clay lumps and friable particles in coarse aggregates, nor 1% by weight of material finer than 75μm (No. 200 ASTM sieve) nor 5% by weight finer than 150μm (No. 100 ASTM sieve).

The shape of particles of the coarse aggregate or coarse fraction of the total aggregate (i.e. particles larger than 10 mm approx.) shall be predominantly angular, sub-angular, sub-rounded or rounded, and shall not contain more than 15% by weight of flakey or elongated particles or of shell or shell fragments.

F.1.5 FINE AGGREGATE

Fine aggregate shall contain no more than 7% by weight of particles passing the 150 μm (ASTM No. 100) sieve, no more than 3% by weight of clay lumps and friable particles, no more than 3% of material finer than the 75μm (ASTM No. 200) sieve, nor more than 0.5% by weight of coal and lignite. (For crushed-stone or other manufactured sand, if the material finer than 150 m consists essentially of dust of fracture, the limit of material finer than 150 m is increased to 15%).

Notwithstanding these allowable limits, it must be recognised that aggregate containing high levels of finer material will exhibit considerably increased demand for mixing water to attain sufficient workability, which will have adverse effects on the achievement of economical concrete of adequate strength and durability.

F.1.6 RECOMMENDED METHOD OF TESTING AGGREGATE FROM NEW OR UNPROVEN SOURCES FOR SUITABILITY FOR USE IN STRUCTURAL CONCRETE:

F.1.6.1 Testing for mechanical strength

Since no simple relationship exists between the results of direct tests on an aggregate and the upper limit(s) of strength of concrete made with the same aggregate, the following procedure is recommended:

Concrete of a range of strength from low (say 15 MPa) to high (say 70 MPa) is made, and compressive strengths are tested to failure at some predetermined age. The extent of failure of the larger aggregate particles (as contrasted with bond failure at the aggregate/cement-paste interface) at any particular strength level will indicate whether or not the aggregate represented by the sample possesses sufficient strength for use in concrete at the strength level.

F.1.6.2 Tests for other characteristics

Petrology: If the aggregate intended for use is derived from apparent rock type for which a satisfactory record of local usage does not exist, or of a type prone to adverse reactions with the typical constituents of cement, samples of the aggregate shall be checked by competent personnel and/or laboratories for likely unsuitability or otherwise due to inherent petrological or chemical character. Such risk of unsuitability may include: aggregate reactivity with alkalis of cement; poor weathering behaviour; high drying shringage; etc.

F.1.7 CRITERIA FOR THE ACCEPTANCE OF CONCRETE BASED ON DESTRUCTIVE COMPRESSIVE STRENGTH TESTING.

The criteria of:

(a) strength level,
(b) acceptable variability of results of strength testing,

for concrete compressive strength specimens sampled from concrete for structural purposes for any major works shall be determined by the Structural Designer, in consultation with a Materials Consultant, recognised laboratory and/or other institution(s) or individual(s), as appropriate to the particualr case. Such strength requirements shall take into account, inter alia the following:

(c) Structural design assumptions concerning the behaviour of concrete under stress;

(d) Possible conflicts between contractual, and engineering criteria or considerations for the definition of "acceptable" concrete. These may, for example, result from different maturities ("ages") at which the concrete is tested for approval, and unverified assumptions concerning strength development of such concrete(s) between such ages. Such conflict is known to exist in at least one of the major CARICOM territories.

(e) The competence and reliability of testing services available to the project.

(f) Prevailing levels of capability of:

- concrete manufacture
- consolidation (compaction),
- workmanship and supervision in concrete construction,

which are expected to be available to the project, since these considerably influence what levels of in-place concrete quality can be achieved in practice.

(g) The consequence of the failure of the concrete to attain the specified strength.

F.1.8 METHODS OF TESTING HARDENED CONCRETE FOR STRENGTH

F.1.8.1 General

ACI 318 is related to a system of concrete quality monitoring in which cylinders moulded from concrete sampled from that being put into a structure are tested compressively to destruction. (While it is recognized that non-destructive, partially destructive, and combined methods have been used for the assessment of concrete strength in structures, these methods have not received widespread acceptance, partly due to insufficient experimental data, and are not proposed for general and/or routine usage at this stage of their development).

ACI 318 makes no provisions for measurement of concrete strength by cubes; it is nonetheless recognized by the Code that cubes as well as cylinders are in use in the CARICOM territories, and are likely to be used for some time.

The following cautions shall be observed in the destructive testing of concrete for compressive strength, using moulded specimens.

F.1.8.2 Testing Equipment

Cylinders are more tolerant of variations in machine behaviour than are cubes, though it is essential that their ends be properly prepared, by capping grinding, or other suitable methods. Equipment designed only for the testing of cylinders (typically North American equipment of inadequate lateral rigidity and symmetry under load for cube testing) shall not be used for the testing of cubes for control or acceptance purposes for major works. Even when such equipment is used for lesser works, it should be regularly referenced against equipment of a more robust nature, designed for cube testing. Equipment designed for cube testing may be used for testing both cylinders and cubes, so that routine checks may be made on the value(s) of correction factors which should be applied for particular concretes.

F.1.8.3 Types of Specimens

Either cylinders or cubes, but not both, shall be used on any project, or major, discrete part thereof. If cube strengths are to be converted to cylinder strengths or vice versa, the conversion factors for concrete of the:

(1) aggregate type and grading
(2) mix proportions
(3) cement type
(4) strength range(s)

proposed to be used should be verified experimentally. In the absence of such information, the relationship:

150 mm diameter cylinder strength
= 0.8 x cube strength (100 mm or 150 mm cube, aggregate particle size 25 mm) (which has been derived from limited experimental verification in the region) shall be used as a guide until more reliable data specific to the particular concretes is available. It must be clearly understood that there is no universally applicable factor which can be used to convert cube strengths to cylinder strengths, but that factor such as concrete strengths and proportions, among other things, affect the relationship.

F.1.8.4 Sizes of Specimens

Specimens shall generally be of the following dimensions:

Cylinders: 300 mm or 1 foot long, 150 mm or 6 inch diameter.
Cubes: 150 mm (or 6 inch) side.

Where the maximum aggregate size does not exceed 25 mm, the specimen sizes may be reduced to the following:

Cylinders: 100 mm diameter, 200 mm long
Cubes: 100 mm side.

Care shall be taken to ensure that metric and Imperial sizes of Specimens are not confusingly used together.

F.1.8.5 Competence of Laboratory Staff

Technical staff in testing organisations shall be properly qualified for the execution of tests in accordance with standard procedures, and shall maintain records of calibration and other checking and servicing of equipment, which records shall be available for inspection.

The foregoing assumes that standard practices such as the capping of cylinders with standard materials against plane machined surfaces, and the other procedures relative to good concrete testing practice are also consistently carried out.

F.1.9 RATE OF GAIN OF CONCRETE COMPRESSIVE STRENGTH WITH TIME

The "normal" ages of testing for standard water-cured, moulded specimens for compressive strength testing shall be 7 and 28 days.

No single generally-applicable relationship exists for the rate of gain of concrete strength between 7 and 28 days; the relationship is dependent, in any particular case, on the following and other factors: cement type, water-cement ratio, aggregate-cement ratio, fine aggregate content, temperature, etc.

Relationships for rates of gain of concrete strength with time identified in temperate climates shall not be used for the prediction of strength at one age from data at another. Estimates of the amount of concrete compressive strength increase after 28 days, developed in non-tropical climates, shall not be used directly to estimate additional structural capacity which will be developed before design loads are applied for structures built under this Code. (This caution shall be observed with particular stringency where members of small cross-section and/or exposed to uncertain curing provisions are concerned).

Records of actual results of strength gain overtime, established over substantial numbers of samples, for concrete of similar porportions and cured under similar conditions, shall be used as the basis for such strength predictions.

In the absence of local data, the following data derived in the Southern Caribbean under controlled conditons, may be used as a guide only:

For the normal ranges of structural concrete (i.e. water-cement ratio by weigth 0.5 to 0.6, aggregate-cement ratio by weight 5.0 to 6.0) the ratio 7-day strength - 28-day strength = 0.6 to 0.7.

For concretes of strengths below 20 MPa 28-day cube strength or above 35 MPa 28-day cube strength, and of aggregate-cement ratio more than 6 or less than 5, experimental verification of strength gain is necessary, particularly if more precise estimation of this relationship (for the range of concretes being used) is important for reasons of structural safety or because of contractual provisions. Particular care shall be exercised in estimating and interpreting such relationships in borderline cases where the acceptance or rejection of a structure depends on such strength estimation(s). In general, richer concretes (i.e. of higher cement content per unit volume) and higher ambient temperatures lead to more rapid early gains of strength, with smaller gains at later ages.

F.1.10 CURING OF SPECIMENS, CURING OF MEMBERS IN THE FIELD

Moulded Compression specimens shall be kept thoroughly moist in the shade (or in a room of 95% minimum relative humidity) and free from vibration or rough handling for the first 24 hours; thereafter they shall be kept in clean standing water in the shade until the time of test, and shall be tested damp (after capping, in the case of cylinders), but free of surface water, grit, fine, etc.

Since moulded specimens cured in water represent potential strength of the concrete sampled, which potential may not be realised in the field due to inadequate curing, small structural members (least linear dimension less than 300 mm) and/or those subjected to sunlight and/or wind, particularly if their structural adequacy is important to the safety of the structure, shall be protected from excessive early moisture loss for at least the first seven(7) days after casting, by covering any concrete faces not protected by formwork by one of the following:

(a) Cover by damp sacking, sealed with taped and securely fastened plastic sheet;
(b) sealed with plastic sheet(as above)only;
(c) coated with a proprietary curing compound in accordance with the manufacture's instructions.

Wrapping (columns and beams) or covering (slabs and other flatwork) with damp sacking only (i.e. not covered by a waterproof membrane) which is wetted intermittently by site personnel or otherwise shall NOT be accepted as a satisfactory method of curing.

Care shall be taken to ensure the flatwork (particularly when of shallow depth, and more so when moisture can be lost from both sides, such as in filler-block composite floor slab construction of suspended floors where relatively thin concrete toppings are used) shall be similarly protected from such moisture loss. It is known that the ponding of slabs is an ideal curing procesure, but it is equally recognized that this is impracticable on many sites.

F.2 Water for Washing Aggregate (particularly Fine Aggregate) and for Mixing Concrete

Such water shall in general be non-saline, clear and free from odour, taste, or visible sediment. In particular water which may contain organic matter such as from peat, humus, swamp vegetation, detritus or waste from meat or fish or other food processing plant, and chicken or other animal waste, shall be avoided. Such water may have adverse effects on, or may entirely prevent the hardening of, concrete. Water which may contain deleterious waste from industrial effluent shall similarly be avoided.

F.2.1 TESTS ON WATER QUALITY WHERE DOUBT EXISTS

Water shall be tested for the presence of contaminants by a recognised laboratory and/or in accordance with procedures relevant to the likely contaminant(s) and risk(s) involved in the particular case. In the assessment of effects on strength, sample concrete may be compared against a control made using potable mains water, rain water or distilled water, particularly in territories such as Antigua where the lower levels of rain experienced and the general absence of surface watercourses can result in higher-than-usual levels of dissolved salts etc. in local waters.

F.3 Cautions pertaining to particular Territories

F.3.1 BARBADOS

Most of the island is covered with coral limestone, of which there are very substantial reserves. This is crushed and screened to provide the coarse aggregate, whereas the sand typically used in concrete is largely siliceous and is from the Scotland District. No problems have been reported with its use.

The coral is not of very high mechanical strength, though records indicate that cube compressive strengths exceeding 44 MPa have been achieved, which is adequate for most normal purposes.

Excessively long mixing times, particularly with semi-dry mixes, should be avoided, to avoid mechanical breaking down of the aggregate particles.

The mechanical properties exhibit considerable uniformity, with one notable exception: aggregate from Mangrove, St. Phillip has recorded Aggregate Crushing Values (37 to 45) which indicate a rock significantly weaker than the Barbados average of 32 to 35. The higher water absorption (7% to 8%) compared with the range of other coral limestone (4% $\pm$ 0.5% typical, one location at 2.2%) also indicates increased risk of corrosion of reinforcement and pre-stressing strand, and aggregate from this source should be avoided, particularly for marine splash-zone concrete.

The values of absorption typical of Barbados coral limestone are higher than is the case for many aggregates. In addition, problems of reinforcement corrosion have emerged on certain engineered buildings near the sea within relatively short periods after construction. These together indicate that some concern regarding durability of sea-side structures in Barbados is appropriate, and the recommendations - (adequate or increased cover to well fixed reinforcement working at moderate tensile stresses, low water-cement ratio, tight formwork, thorough compaction and curing) for durability for marine works should be observed with particular care in such constructions.

The relatively high absorptive capacity of the coral coarse aggregate should be considered in the manufacture of concrete when it is apparent that a supply of aggregate is not in a saturated and surface-dry conditon. Such aggregates will absorb water from the mix, with a resulting loss of workability. Depending on the rate of absorption, some unexpected increase of hardened compressive strength may be observed due to an effective internal reduction of the water-cement ratio after placement and compaction. There is as yet insufficient data available for this aggregate to propose quantitative factors for attempting to take cautious advantage of this mechanism in mix design.

F.3.2 DOMINICA

No deleterious or adverse reports have been reported for aggregates for the following sources, which represent the main sources in use in 1984.

Roseau River, gravel deposit extracted from river-bed. concrete compressive strengths exceed 45 MPa (specimen type not specified).

Belfast River, basaltic river gravel deposits.

Government Stock Farm, pumice deposit, high absorption, used in the manufacture of concrete masonry units.

Canefield (new quarry - data not yet available).

Ravine Gabriel, Colihaut, grey fractured rock deposit of the basalt group. Drilled, blasted, crushed and screened. Concrete strengths exceed 45 MPa.

Pointe Ronde, sand deposit.

Melville Hall River, dark basaltic river gravel deposits.

No particular cautions are therefore appropriate.

F.3.3 GUYANA

F.3.3.1 Coarse Aggregate

(i) The major traditional sources are centered around Bartica and are either granites, dolerites or gneisses. The quarries active in 1985 were:

St. Mary's - dolerite, very hard, very good quality

Teperu - massive granite, elongation and flakiness indices usually slightly above normally allowable limits

Baracara - massive coarse-grained muscovite granite of adequate quality, flakiness not as severe as Teperu.

Cautions:

(i) With the granites, Flakiness Index values (BS 812: Part I:1975) as high as 60 have been obtained, and this is excessive; clearly, careful selection is necessary. "The presence of elongated or flaky particles in excess of 10 to 15 percent of the weight of the coarse aggregate is generally considered undesirable, but no recognized limits are laid down" (A.M. Nelville, Properties of Concrete, 3rd Ed., pg. 126). The Ministry of Works in Guyana was a maximum allowable Flakiness Index of 35.

(ii) Since 1983-84, lateritic gravels (s.g. circa 3.0) from small lateritic gravel deposits have been used in the Rupununi, the North-West District, Mabura (Central Essequibo), and the Eberoabo Savannahs. Washing and screening is necessary; concrete strengths have exceeded 21 MPa (cube). Not much used, use likely to be greatly increased in future.

F.3.3.2 Fine Aggregate

Timehri White Sand covers approximately 30% of the total land space. More than 99% pure silica, no strength or durability problem, but typical grading is deficient in particles between 5 mm and 1.18 mm, so care may be necessary in obtaining workable overall gradings with coarse aggregates.

F.3.4 ST. LUCIA

The following quarries are in operation:

(a) Ferrands Quarry
(b) Quarry Products
(c) Giraudy Quarry
(d) Northrock Ltd.
(e) Monrose Quarry
(f) Hess (Oil Co.) Quarry

Sources:

(a) massive rock
(b,c,d) fractured rock
(e) boulders
(f) not stated

All parent rock types are suitable for concrete; wide range of sizes available at most quarries; no data from laboratory testing of mechanical or chemical characteristics available. Pumice is also used as fine aggregate with all of the above aggregates for reinforced concrete of over 24.5 MPa 28-day compressive strength (specimen type not specified). All aggregates (a) to (f) are therefore sufficiently strong for such concrete, with concrete strengths of over 31 MPa possible by both (a) and (e). Concrete strengths of over 28 MPa have been achieved by (c).

Cautions:

All aggregates are unwashed; some traces of silt therefore exist on purchased aggregates. Caution is necessary regarding the risk of one-size-tendency coarse aggregate used with some beach-sand-type fine aggregates, leading to grading gaps in the combined aggregate. Particularly with angular crushed aggregates, this may lead to concrete of low cohesion, i.e. prone to segregation and likely to exhibit large and relatively sudden changes of workability with small changes in water content.

Northrock Limited

Most commonly used in north of the island. Excellent aggregate for all purposes, but size variations as high as 20% from specified size have been noted.

Quarry Products

Particles can sometimes be flaky and flat instead of angular. Selectiveness is necessary.

Giraudy Quarry

Uses same quarry site as Quarry Products, but size variation from specified size is high - over 20%

Ferrands Quarry and Hess Quarry

Both extract from a rock outcrop within a 1.6 Km radius. Very little variation exists in the aggregates produced. Special precaution is necessary with "3/16 inch dust" because of its high silt conent.

Monrose Quarry

No additional cautions reportedly necessary.

F.3.5 ST. VINCENT (including the Grenadines of St. Vincent)

Few tests on local aggregates have been done; data on properties is scarce. All coarse aggregates are crushed and screened from volcanic parent rock. Obtaining sufficient fine aggregate (usually beach sand) can be difficult; major projects should arrange adequate supplies beforehand. Coarse aggregates can contain undesirable levels and sizes of oversize particles. Careful screening is necessary to ensure compliance with workable specifications and to prevent problems regarding reinforced concrete monolithicity and integrity of cover. Aggregate strength is adequate; 7-day compressive strengths exceeding 31 Mpa are readily obtainable.

In the Grenadines, reinforcement corrosion risk is increased by the brackish mixing water typically available. The use of rain or transported fresh water is strongly recommended, particularly for concrete in damp or underground locations.

The aggregate supply situation on the main island is expected to improve with the introduction of better crushing plant and the completion of a Quarry Development survey, expected to identify inter alia suitable fine aggregate sand sources in the Soufriere region.

F.3.6 TRINIDAD AND TOBAGO

F.3.6.1 Trinidad

The main aggregate types used for engineered structural concrete are

Guanapo, and
Melajo quartzitic gravels
Northern Range Limestone (crushed rock)

These aggregates are strong, of low porosity, durable, and relatively inert. No limitations of a directly petrologic origin have emerged in their use. There is need to monitor the flakiness of some Northern Range crushed limestone which in some instances attain visibly undesirable levels.

Few tests results exist for aggregates (or concrete made therefrom) from sources south of the Caroni River. HOWEVER, Guaracara Limestone (so-called "yellow" limestone) has been used as a crushed aggregate for concrete for residential and larger buildings for some years, with fair results. No records exist of its successful use in heavy civil works; laboratory research regarding aggregate properties is in progress, and some results are encouraging.

San Fernanado Hill "gravel" (argillite) has been used in largely minor works for many years; its use is now illegal.

Porcellanite has been used as a concrete aggregate for domestic and minor construction in the extreme south of the island, but

- low mechanical properties,
- variable properties,
- high permeability, and
- poor weathering performance

make this material unsuitable for engineered concrete.

Electric-arc furnace slag from steel smelting has recently become available and may be a potential future aggregate. Little if any data on experience of its use in engineered concrete exists to date, and the Code cannot now make any responsible recommendations concerning its use.

F.3.6.2 Tobago

No cautions have been reported for aggregate used in Tobago. Users are however recommended to established aggregate production facilities (at the present time - 1984 - those at Godsborough and Green Hill) where better conformity to predictable gradings is likely.

Tobago coral limestone covers approximately 20% of the land area in the south west of the island. Its use is not recommended for structural concrete, particularly since its performance is not proven, and the insular character of Tobago makes corrosion of reinforcement in near sea concrete a substantial risk over much of the island.

F.3.7 GRENADA

The island is composed mainly of massive discrete lava flows and pyroclastic rocks, with some development of localized fossiliferous stratified sedimentary beds also evident. Substantial disintegration and weathering is evident in some deposits.

There are three(3) principal quarries in Grenada:

(1) Queen's Park (St. George's) Est. reserves
5.0 M tonnes red gravel
1.9 M tonnes black gravel

red and black cinder deposits or "gravel" (scoriaceous basaltic cinders), excavated by bulldozer; no crusher, sieving only, approx. 28 tonnes/hr.

(2) Telescope Point (north) Est. reserves 3.9 M tonnes

massive basalt; blasted, sledged, and handpicked and fed to crusher (rated lot/hr, approx. 5t/hr produced).

(3) Mount Hartmann (St. George's) Est. reserves +2.0 M tonnes

fractured basalt, fed to jaw crusher for both primary and secondary crushing; rated 60t/hr., achieved 30t/hr average.

No details available on typical gradings.

Main uses:

Crushed stone - airport runway and parking, wearing courses for roads.

Red and black gravels - reinforced concrete, and blockwork.

While petrographically the red gravels are similar to the basalts, both red and black gravels exhibit high absorption (6.4%), and though there are no adverse effects reported, caution is recommended in the use of these aggregates in reinforced concretes intended for marine use (particularly in the splash zone) or in other aggressive service environments. Comparison of other properties with the Mt. Hartmann basalt also suggests aggregates which are not of prime quality. Experienced local contractors report better concrete cube strength results if the red gravel for use is wetted the day before.

Cube compressive strengths of 37.5 MPa (5450 psi) and 45.2 MPa (6560 psi) respectively have been obtained using red gravel and Mt. Hartmann, material, but based on past test results an experienced local consultant specifies 20.9 MPa (3000 psi) cube strength where better ("Barbadian") standards of control are expected. Where lower standards are likely, the consultant still specifies 20.9 MPa (3000 psi) cube strength mixes, but uses 16.5 Mpa (2400 psi) cube strength in design.

It seems clear that use of the red and black gravel is not advisable where concrete of the highest quality is required, particularly where durability under severe conditions of exposure is intended. The usual cautions concerning grading, quantity of mixing water, and curing etc. still apply.

F.4 General Cautions

F.4.1 CONCRETE CONSTRUCTION PRACTICES

The importance of adequately supervised good practices in the attainment of durable concrete cannot be over-emphasized; practicable designs, careful placement and compaction, rounded corners, leak-free smooth-faced formwork and proper curing all contribute to increased durability. In extreme conditions, protective or isolating membranes or special surfce treatments may also be required.

F.4.2 MINIMUM CEMENT CONTENTS IN CONCRETE INTENDED FOR AGGRESSIVE LOCATIONS

ACI 318-43 does not prescribe minimum allowable cement contents for aggressive locations. This limitation may adversely affect concrete durability, and users are referred to CP110: Part I:1972 Tables 48, 49 (pp 98,99) and Building Research Establishment Digest 250 "Concrete in sulphate-bearing soils and ground-waters" for further guidance regarding the attainment of adequate durability.

F.4.3 MAXIMUM SIZE OF AGGREGATE PARTICLES

Clause 3.3.3 (c) of ACI 318-83 shall only be waived under exceptional circumstances when the Engineer is satisfied that sufficient aggregate particles are present in the concrete mix which are small enough to pass into and fill the space between, and cover to, reinforcement. This caution shall be enforced. This particular stringency in concrete intended for marine service, especially splash-zone concrete.

F.4.4 HAND-BROKEN COARSE AGGREGATE

Hand-broken aggregate (produced in some of the smaller islands) is in general not suitable for concrete manufacture, as it is largely one size-range, and is unlikely to possess a suitable grading.

F.4.5 AGGREGATES NOT COMPLYING WITH SPECIFICATIONS

Particular attention is drawn to clauses 3.3.2 of ACI 318-83, which may be particularly relevant in certain locations.

FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

$Feet^2$ (ft^2) x 0.0929030	= $Meters^2$ (m^2)
Acres x 4046.86	= $Meters^2$ (m^2)
$Miles^2$ (mi^2) x 2.58999	= $Kilometers^2$ (km^2)

VOLUME

$Feet^3$ (ft^3) x 0.02831685	= $Meters^3$ (m^3)
$Feet^3$ (ft^3) x 28.31685	= Liters**
$Yards^3$ (yd^3) x 0.764555	= $Meters^3$ (m^3)
Acre feet x 1233.48	= $Meters^3$ (m^3)
Gallons (gal) x 0.00454609	= $Meters^3$ (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

$Feet^3$ per second (ft^3/s) x 0.02831685	= $Meters^3$ per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= $Meters^3$ per second (m^3/s)
Acre-feet per day x 0.0142764	= $Meters^3$ per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= $Meters^3$ per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 ($lb\text{-}s/ft^2$) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

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* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

NOTES

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 7A

Structural Design Requirements
STRUCTURAL STEEL

LIMIT STATES DESIGN

1985

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 7A
STRUCTURAL STEEL

LIMIT STATES DESIGN

Caribbean Community Secretariat
Georgetown
Guyana

1985

PART 2

SECTION 7

TABLE OF CONTENTS

FOREWORD

This Section is divided into three parts:

Section 7A - Limit States Design
Section 7B - Working Stress Design
Section 7C - Commentary

The work on this Section was carried out by Messrs. Adams, Kennedy and Kulak of the Department of Civil Engineering, University of Alberta, under a contract with the Caribbean Community. It is suggested that comments on the alternative design methods, or on any of the design details recommended, be sent to the authors.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

Each of the Sections 7A, Limit States Design; 7B, Working Stress Design; and 7C, Commentary have been numbered individually so as to provide continuity between sections. The number and digit corresponding to the Part and Section in the Part (2.7) have been omitted. The numbers that remain correspond to the sub-sections and articles.

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 7A

STRUCTURAL STEEL

LIMIT STATES DESIGN

Disclaimer

The authors of this publication, Messrs. Adams, Kennedy and Kulak, do not warrant the contents and the suitability of this material for general or particular use, nor is there any implication that there is freedom from infringement of patents. The design of structures is within the scope of expertise of a competent licensed professional. Such user of this Standard assumes all liability arising from its use.

Contents

Preface

This is the first edition of the Standard, "Structural Steel for Buildings", Part 2.7A - Limit States Design developed for the Caribbean Uniform Building Code. A first edition of a parallel Standard Steel Structures for Buildings Part 2.7B - Working Stress Design has also been produced for the Caribbean Uniform Building Code. The third document in this sequence is Steel Structures for Buildings Part 2.7C - Commentary.

The Standard is based on the limit states design philosophy. In limit states design the designer compares the effect of factored loads to the minimum likely developable resistance. Limit states define the various types of collapse and unserviceability that are to be avoided and the object of design is to keep the probability of a limit state being reached below a certain value previously established. This is achieved by applying load factors to the specified loads and resistance factors to the specified resistances. By the use of different factors for different loads and resistances a more uniform level of safety is obtained.

The clauses relating to fabrication and erection should serve to remind designers that design and construction are part of the same sequence. The review of construction by competent engineers is of equal importance to the design.

When designing structures under this Standard no use shall be made of the companion Standard, Steel Structures for Buildings - Working Stress Design. The Commentary clarifies the intent of various provisions of this Standard. The publications listed as references provide the extensive background used for the development of the Standard and its technical requirements.

This Standard sets out minimum requirements for steel structures as outlined in the Scope, and it is expected that it will only be used by engineers competent in the field. Although the intended primary application of the Standard is stated in the Scope clause, it is important to note that it remains the responsibility of the user of the Standard to judge its suitability for his particular purpose.

The Standard is based in large measure on CSA Standard CAN3-S16.1-M84 Steel Structures for Buildings - Limit States Design and on an October 1983 draft of CSA Standard CAN3-S136-M84 Cold Formed Steel Structural Members and reflects the latest research on steel building structures. The Canadian Standards Association assumes no responsibility for its content.

Although every effort has been made in writing and proofreading this Standard to ensure that all information is accurate and that all numerical values are correct some errors may have been overlooked. Users are requested to bring any such errors found to the attention of the authors.

Reference Publications

This Standard refers to the following publications. The years shown indicate the latest issues available at the time of printing.

CSA Standards
CAN3-A23.1-M77,
Concrete Materials and Methods of Concrete Construction;

CAN3-A23.3-M84,
Code for the Design of Concrete Structures for Buildings

B95-1962,
Surface Texture (Roughness, Waviness, and Lay);

G28-1968,
Carbon-Steel Castings for General Application;

G38-1953,
Heavy Steel Shaft Forgings;

CAN3-G40.20-M81,
General Requirements for Rolled or Welded Structural Quality Steel;

CAN3-G40.21-M81,
Structural Quality Steels;

G189-1966,
Sprayed Metal Coatings for Atmospheric Corrosion Protection;

S37-M1981,
Antenna Towers and Antenna Supporting Structures;

S136-M1984,
Cold Formed Steel Structural Members;

W47.1-1973,
Certification of Companies for Fusion Welding of Steel Structures;

W48.1-M1980,
Mild Steel Covered Arc-Welding Electrodes;

W48.3-1976,
Low-Alloy Steel Arc-Welding Electrodes;

W48.4-M1980,
Solid Mild Steel Electrodes for Gas Metal-Arc Welding;

W48.5-1970,
Mild Steel Electrodes for Flux Cored Arc Welding;

W48.6-M1980,
Bare Mild Steel Electrodes and Fluxes for Submerged-Arc Welding;

W55.3-1965,
Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings;

W59-1977,
Welded Steel Construction (Metal-Arc Welding);

ANSI/ASTM† Standards
A27-80,
Mild- to Medium-Strength Carbon-Steel Castings for General Application;

A36-77a,
Structural Steel;

A108-79,
Steel Bars, Carbon, Cold-Finished, Standard Quality;

A148-80,
High Strength Steel Castings for Structural Purposes;

A242-81,
High Strength Low-Alloy Structural Steel;

A283-81,
Low and Intermediate Tensile Strength Carbon Steel Plates of Structural Quality;

A307-80,
Carbon Steel Externally and Internally Threaded Standard Fasteners;

A325-82,
High-Strength Bolts for Structural Steel Joints;

A325 M-82,
High-Strength Bolts for Structural Steel Joints (Metric);

A441-79,
High-Strength Low-Alloy Structural Manganese Vanadium Steel;

A446-76,
Steel Sheet, Zinc Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality (Grades A, B, C, D and F);

A486-74 (Reapproved 1980),
Steel Castings for Highway Bridges;

A490-82,
Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength;

A490 M-82,
High-Strength Steel Bolts, Classes 10.9 and 10.9.3 for Structural Steel Joints (Metric);

A514-77,
High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding;

A521-76,
Steel, Closed-Impression Die Forgings for General Industrial Use;

A525-79,
Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, General Requirements;

A570-79,
Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality;

A572-81a,
High-Strength Low Alloy Columbium-Vanadium Steels of Structural Quality;

A588-80a,
High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick;

A606-75,
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength, Low-Alloy, with Improved Corrosion Resistance;

A607-75,
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy Columbium and/or Vanadium;

A611-72,
Steel, Cold-Rolled Sheet, Carbon, Structural (Grades A, B, C and D);

A668-79a,
Steel Forgings, Carbon and Alloy, for General Industrial Use;

A715-81,
Sheet Steel and Strip, Hot-Rolled, High-Strength Low-Alloy, with Improved Formability;

CGSB# Standards
1-GP-14M-1979,
Primer, Red Lead in Oil;

1-GP-40M-1979,
Primer, Structural Steel, Oil Alkyd Type;

1-GP-81M-1978,

Primer, Alkyd, Air Drying and Baking, for Vehicles and Equipment;

1-GP-140M-1978,
Primer, Red Lead, Iron Oxide, Oil Alkyd Type;

1-GP-166M-1979,
Primer, Basic Lead Silico-Chromate, Oil Alkyd Type;

CISC/CPMA§ Standards
1-73a,
A Quick-Drying One-Coat Paint For Use On Structural Steel;

2-75,
A Quick-Drying Primer For Use On Structural Steel;

Canadian Institute of Steel Construction
Code of Standard Practice for Structural Steel;

CSSB1†† Standards
101 M-78,
Zinc Coated Structural Quality Steel Sheet for Roof and Floor Deck;

National Building Code of Canada, 1980;

Metric Values for Use with the National Building Code, 1980;

SSPC Specifications**
PS 12.00-68T,
Guide to Zinc-Rich Coating Systems;

PT 3-64,
Basic Zinc Chromate - Vinyl Butyral Washcoat;

SP 2-63,
Hand Tool Cleaning;

SP 3-63,
Power Tool Cleaning;

SP 4-63,
Flame Cleaning of New Steel;

SP 5-63,
White Metal Blast Cleaning;

SP 6-63,
Commercial Blast Cleaning;

SP 7-63,
Brush-Off Blast Cleaning;

SP 10-63T,
Near-White Blast Cleaning;

Structural Stability Research Council
Guide to Stability Design Criteria for Metal Structures.

* American National Standards Institute.
† American Society for Testing and Materials.
‡ Canadian General Standards Board.
§ Canadian Institute of Steel Construction/Canadian Paint Manufacturers' Association.
†† Canadian Sheet Steel Building Institute
** Steel Structures Painting Council.

Structural Steel - Limit States Design

Where reference in this Standard is made to National Standards of Canada, National Standards of other countries may be used when approved by the Regulatory Authority.

1. **Scope**

1.1 This Standard provides rules and requirements for the design, fabrication, and erection of steel structures for buildings where the design is based on limit states. The term "steel structures" relates to structural members and frames which consist primarily of structural steel components, including the detail parts, welds, bolts, or other fasteners required in fabrication and erection. Composite construction, defined as construction which depends upon the participation of structural elements of steel and other materials in resisting loads and forces jointly with structural steel members, is permitted under this Standard. Clause 14 applies to the design of structural members cold formed to shape from carbon or low alloy steel sheet, strip or plate up to 25 mm in thickness and intended for load carrying purposes in buildings.

1.2 Where reference is made to other publications, such reference shall be considered to refer to the latest edition or any revision thereto approved by the organization issuing that publication.

1.3 When designing structures under this Standard, no use shall be made of the companion standard, Steel Structures for Buildings - Working Stress Design.

2. **Application**

2.1 This Standard applies unconditionally to steel structures for buildings except as noted in Clause 2.2.

2.2 Supplementary rules or requirements may be necessary for:

(a) Unusual types of construction

(b) Mixed systems of construction; and

(c) Steel structures which

(i) Have great height or spans;

(ii) Are required to be movable or be readily dismantled;

(iii) Are exposed to severe environmental conditions, or possible severe loads such as those resulting from vehicle impact or chemical explosion;

(iv) Are required to satisfy aesthetic, architectural, or other requirements of a non-structural nature;

(v) Employ materials or products not listed in Clause 5;

(vi) Have other special features that could affect design, fabrication, or erection.

2.3 A rational design based on theory, analysis, and engineering practice, acceptable to the Regulatory Authority, may be used in lieu of the formulae provided in this Standard. In such cases the design shall provide nominal margins (or factors) of safety at least equal to those intended in the provisions of this Standard (see Appendix E).

3. Definitions and Symbols

3.1 Definitions

The following definitions apply to this Standard:

General

Approved means approved by the Regulatory Authority;

Regulatory Authority means a government Ministry, Department, Board, Agency, or Commission that has responsibility for regulating, by statute, the use of products, materials, or services;

Limit states means those conditions of a structure in which the structure ceases to fulfil the function for which it was designed. Those states concerning safety are called the ultimate limit states and include exceeding of load-carrying capacity, overturning, sliding, fracture and fatigue. Those states which restrict the intended use and occupancy of the structure are called serviceability limit states and include deflection, vibration and permanent deformation.

Loads

Gravity load (newtons) is equal to the mass of the object (kilograms) being supported multiplied by the acceleration due to gravity, g (9.81 m/s^2);

Specified loads (D, L, Q and T) means those loads prescribed by the Regulatory Authority (see Clause 7.1);

Factored load means the product of a specified load and its load factor.

Factors

Load factor, α, means a factor, given in Clause 7.2, applied to a specified load for the limit states under consideration to take into account the variability of the loads and load patterns and analysis of their effects;

Load combination factor, ψ, means a factor, given in Clause 7.2, applied to the factored loads other than dead load to take into account the reduced probability of a number of loads from different sources acting simultaneously;

Importance factor, γ, means a factor, given in Clause 7.2, applied to factored loads to take into account the consequences of collapse as related to the use and occupancy of the structure;

Resistance factor, ϕ, means a factor, given in the appropriate clauses of this Standard, applied to a specified material property or the resistance of a member, connection or structure, which, for the limit state under consideration, takes into account the variability of material properties, dimensions, workmanship, type of failure, and uncertainty in prediction of member resistance. To maintain simplicity of design formulae in this Standard, the type of failure and uncertainty in prediction of member resistance have been incorporated in the expressions of member resistance. (See Appendix E for a more detailed discussion.)

Resistance

Resistance, R, of a member, connection or structure is calculated in accordance with this Standard based on the specified material properties and nominal dimensions;

Factored resistance, ϕR, means the product of the resistance and the appropriate resistance factor.

Tolerances

Camber means the deviation from straightness of a member or any portion of a member with respect to its major axis. Frequently camber is specified and produced in a member to compensate for deflections that will occur in the member when loaded. (See Clause 6.2.2.) Unspecified camber is sometimes referred to as bow;

Sweep means the deviation from straightness of a member or any portion of a member with respect to its minor axis;

Mill tolerances means variations allowed from the nominal dimensions and geometry with respect to cross-sectional area, non-parallelism of flanges and out-of-straightness such as sweep or camber in the product as manufactured and are given in CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel;

Fabrication tolerances means tolerances allowed from the nominal dimensions and geometry such as the cutting to length, finishing of ends, cutting of bevel angles, and for fabricated members, out-of-straightness such as sweep and camber (see Clause 26);

Erection tolerances means tolerances related to the plumbness, alignment, level, of the piece as a whole. The deviations are determined by considering the locations of the ends of the piece with respect to the positions stipulated on the drawings.

Note: Additional definitions are found in the Standard particularly in Clauses 14, 17 and 18.

3.2 **Symbols**

The following symbols are used throughout this Standard. Deviations from them, and additional nomenclature, are noted where they appear. Dimensions in mm and forces in newtons are assumed unless otherwise noted.

A Area

A_b Cross-sectional area of one bolt based on nominal diameter

A_f Flange area

A_g Gross area

A_m Area of fusion face

A_n Critical net area

A_r Area of reinforcing steel

A_s Area of steel section including cover plates; area of bottom (tension) chord of steel joist; area of stiffener or pair of stiffeners

A_{sc} Area of steel shear connector

A_w Web area; shear area; effective throat area of weld

a Centre-to-centre distance between transverse web stiffeners; depth of concrete compression zone

a' Length of cover plate termination

a/h Aspect ratio, ratio of distance between stiffeners to web depth

B Bearing force in a member or component under specified load

B_f Bearing force in a member or component under factored load

B_r Factored bearing resistance of a member or component

b Width of stiffened or unstiffened compression elements; design effective width for a concrete slab of a composite beam

C Compressive force in a member or component under specified load; axial load

C_e Euler buckling strength = $1\ 970\ 000A/(KL/r)^2$, (newtons)

C_f Compressive force in a member or component under factored load; factored axial load

C_r Factored compressive resistance of a member or component; factored compressive resistance of steel acting at the centroid of that part of the steel area in compression

C_r' Compressive resistance of concrete acting at the centroid of the concrete area in compression

C_w Warping torsional constant

C_y Axial compressive load at yield stress

D Outside diameter of circular sections; diameter of rocker or roller; also stiffener factor

d Depth; overall depth of a section; diameter of bolt or stud

d_b Depth of beam

E Elastic modulus of steel (200 000 MPa assumed)

E_c Elastic modulus of concrete

e End distance; lever arm between the compressive resistance, C_r, and tensile resistance, T_r

e' Lever arm between the compressive resistance, C_r', of concrete and tensile resistance, T_r, of steel

F Strength or stress (MPa unless noted)

F_{cr} Critical plate buckling stress

F_s Ultimate shear strength

F_{sr} Allowable range of stress in fatigue

F_{st} Factored axial force in the stiffener

F_u Specified minimum tensile strength

F_y Specified minimum yield stress, yield point or yield strength

F_{yr} Specified yield strength of reinforcing steel

f_c' Specified compressive strength of concrete at 28 days

G Shear modulus of steel (77 000 MPa assumed)

g Transverse spacing between fastener gauge lines (gauge distance); acceleration due to gravity (m/sec^2)

h Clear depth of web between flanges; height of stud

h_d Depth of cellular steel deck

I Moment of inertia

I_e Effective moment of inertia

I_g Moment of inertia of cover-plated section

I_s Moment of inertia of the steel section

I_t Moment of inertia of the transformed composite section

J St. Venant's torsion constant

K Effective length factor

KL Effective length

k Distance from outer face of flange to web toe of fillet of I-type sections

k_b Buckling coefficient

k_v Shear buckling coefficient

L Length

L_c Length of channel shear connector

L_{cr} Maximum unbraced length adjacent to a plastic hinge

M Bending moment in a member or component under specified load

M_f Bending moment in a member or component under factored load

M_{f1} Smaller factored end moment of a beam-column; factored bending moment at a point of concentrated load

M_{f2} Larger factored end moment of a beam-column

M_p Plastic moment = ZF_y

M_r Factored moment resistance of a member or component

M_{rc} Factored moment resistance of composite beam

M_u Moment resistance of a member subject to lateral buckling

M_y Yield moment = SF_y

m Number of faying surfaces or shear planes in a bolted joint, equal to 1 for bolts in single

λ Non-dimensional slenderness ratio used in column formula

μ Coefficient related to the slip resistance of a bolted joint

ρ Mass density of water, kg/m^3; Constant which depends upon Poisson's ratio for steel and concrete

τ Empirical coefficient used to account for additional strength from a triaxial stress state developed in a concrete filled column

τ' Empirical coefficient used to account for additional strength from a triaxial stress stated developed in a concrete filled column

ϕ Resistance factor (see definition under Factors, in Clause 3.1)

ϕ_w Resistance factor for welds

ψ Load combination factor

ω Coefficient used to determine equivalent uniform bending effect in beams or beam-columns

4. Drawings

4.1 Design Drawings

4.1.1 Design drawings shall be drawn to a scale adequate to convey the required information. The drawings shall show a complete design of the structure with members suitably designated and located, including such dimensions and detailed description as necessary to permit the preparation of shop details and erection diagrams. Floor levels, column centres, and offsets shall be dimensioned.

4.1.2 Design drawings shall designate the design standards used, show clearly the type or types of construction as defined in Clause 8 to be employed, and shall designate the material or product Standards applicable to the members and details depicted (see Clause 5). Drawings shall be supplemented by data concerning the governing loads, shears, moments, and axial forces to be resisted by all members and their connections when needed for the preparation of shop details. (See also Clause 20.1.2.)

4.1.3 Where high-strength bolted joints are required to resist shear between connected parts, the design drawings shall indicate the type of joint, slip-resistant (friction) or bearing, to be provided (see Clause 23).

4.1.4 If required, camber of beams, girders, and trusses shall be called for on the design drawings.

4.2 **Shop Details**

Shop details giving complete information necessary for the fabrication of the various members and components of the structure, including the required material and product standards and the location, type and size of all mechanical fasteners and welds, shall be prepared in advance of fabrication, and submitted for approval when so specified. Shop details shall distinguish clearly between mechanical fasteners and welds required for shop fabrication and those required in the field.

4.3 **Erection Diagrams**

Erection diagrams shall show the principal dimensions of the structure, piece marks and sizes of the members where necessary for approval, elevation of the column bases, all necessary dimensions and details for setting anchor bolts and all other information necessary for the assembly of the structure. (See also Clause 20.1.3.)

5. Material: Standards and Identification

5.1 Standards

5.1.1 **General**

Acceptable material and product standards and specifications (latest editions) for use under this Standard are listed in Clauses 5.1.2 to 5.1.8 inclusive. Materials and products other than those listed may also be used if approved. Approval shall be based on published specifications which establish the properties, characteristics, and suitability of the material or product to the extent and in the manner of those standards which are listed.

5.1.2 **Structural Steel**

CSA G40.21-M,
Structural Quality Steels

5.1.3 **Sheet Steel**

ASTM A570,
Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality.

Other standards for structural sheet are listed in Clause 14. Only structural quality sheet standards which specify chemical composition and mechanical properties will be acceptable for use in the other clauses of this Standard. Mill test certificates which list the chemical composition and the mechanical properties shall be available, upon request, in accordance with Clause 5.2.1(a).

5.1.4 **Cast Steel**

CSA G28,
Carbon-Steel Castings for General Application;

ASTM A27,
Mild- to Medium-Strength Carbon-Steel Castings for General Application;

ASTM A148,
High-Strength Steel Castings for Structural Purposes;

ASTM A486,
Steel Castings for Highway Bridges.

5.1.5 **Forged Steel**

CSA G38,
Heavy Steel Shaft Forgings;

ASTM A521,
Steel, Closed-Impression Die Forgings for General Industrial Use;

ASTM A668,
Steel Forgings, Carbon and Alloy, for General Industrial Use.

5.1.6 **Bolts**

ASTM A307,
Carbon Steel Externally and Internally Threaded Standard Fasteners;

ASTM A325,
High-Strength Bolts for Structural Steel Joints;

A325M,
High-Strength Bolts for Structural Steel Joints (Metric);

A490,
Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength

A490M,
High-Strength Steel Bolts, Classes 10.9 and 10.9.3 for Structural Steel Joints (Metric).

Note: Before specifying metric bolts, the designer should check on their current availability in the quantities required.

5.1.7 **Welding Electrodes**
CSA W48.1
Mild Steel Covered Arc-Welding Electrodes;

CSA W48.3,
Low-Alloy Steel Arc-Welding Electrodes;

CSA W48.4,
Solid Mild Steel Electrodes for Gas Metal-Arc Welding;

CSA W48.5,
Mild Steel Electrodes for Flux Cored Arc Welding;

CSA W48.6,
Bare Mild Steel Electrodes and Fluxes for Submerged-Arc Welding.

5.1.8 **Studs**
ASTM A108,
Steel Bars, Carbon, Cold-Finished, Standard Quality, Grades 1015 and 1018.

5.2 **Identification**

5.2.1 **Methods**
The materials and products used shall be identified as to specification, including type or grade, if applicable, by one of the following means, except as provided in Clauses 5.2.2 and 5.2.3:

(a) Mill Test Certificates or Producer's Certificates satisfactorily correlated to the materials or products to which they pertain;

(b) Legible markings on the material or product made by its Producer in accordance with the applicable material or product standard.

5.2.2 **Unidentified Structural Steel**
Unidentified structural steel shall not be used, unless approved by the building designer. If the use of unidentified steel is authorized, F_y shall be taken as 210 MPa and F_u shall be taken as 380 MPa.

5.2.3 **Tests to Establish Identification**
Unidentified structural steel may be tested to establish identification when permitted by the building designer. Testing shall be done by an

approved testing agency in accordance with CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel. The test results, taking into account both mechanical properties and chemical composition, shall form the basis for classifying the steel as to specification.

5.2.4 **Affidavit**

The fabricator, if requested, shall provide an affidavit stating that the materials and products which he has used in fabrication conform to the applicable material or product standards called for by the design drawings or specifications.

6. Design Requirements

6.1 **General**

As set out in this Standard, steel structures for buildings shall be designed to be serviceable during the useful life of the structure and safe from collapse during construction and during the useful life of the structure. Limit states define the various types of collapse and unserviceability that are to be avoided; those concerning safety are called the ultimate limit states (strength, overturning, sliding, fatigue) and those concerning unserviceability are called the serviceability limit states (deflections, vibration, permanent deformation). The object of limit state design calculations is to keep the probability of a limit state being reached below a certain value previously established for the given type of structure. This is achieved in this Standard by the use of load factors, applied to the specified loads (see Clause 7) and resistance factors applied to the specified resistances (e.g., Clause 13). These factors replace the single safety factors used in other design standards. The various limit states are set out in this Clause. Some of these relate to the specified loads and others to the factored loads. Camber, provisions for expansion and contraction and corrosion protection are further design requirements related to serviceability and durability. All limit states shall be considered in the design.

6.2 **Requirements Under Specified Loads**

6.2.1 **General**

Steel members and frames shall be proportioned so that the serviceability limit state requirements computed on the basis of specified material properties and nominal dimensions are not exceeded by the actions of the specified loads.

6.2.2 **Deflections**

6.2.2.1 Steel members and frames shall be proportioned so that deflections are within acceptable limits for the nature of the materials to be supported and the intended use and occupancy.

6.2.2.2 In the absence of a more detailed evaluation see Appendix H for recommended values for deflections.

6.2.2.3 Roofs shall be designed to withstand any loads likely to occur as a result of ponding.

6.2.3 **Rain Loads**

6.2.3.1 Any roof which can accumulate water shall be designed for the load that can result from a 24 hour rainfall, or such greater load prescribed by the Regulatory Authority, on the horizontal projected area of the roof whether or not the surface is provided with drainage, such as rain water leaders. This applies particularly to large flat roofs in areas of heavy rainfall.

6.2.3.2 The distribution of rain load shall be determined taking into account the shape of the roof, including camber, with or without creep deflection due to dead load in the case of composite structures, and any loads likely to occur as a result of ponding.

6.2.3.3 The amplification of deflections due to ponding shall be considered. This amplification need not be considered provided that the roof stiffness meets the criterion:

(a) For one-way system of roof beams or decking simply supported on rigid supports

$$EI_b > 2\,\rho g S_b\left(\frac{L_b}{\pi}\right)^4$$

(b) For two-way system of roof joists on girder

$$\frac{2\rho g S_j}{EI_j}\left(\frac{L_j}{\pi}\right)^4 + \frac{2\rho g S_g}{EI_g}\left(\frac{L_g}{\pi}\right)^4 < 1$$

where

E = modulus of Elasticity
I = moment of inertia of the beam, joist, girder or unit width of decking as applicable
L = span of the beam, joist or girder as

applicable
S = spacing of the beam, joist, girder or unit width of decking as applicable
ρ = mass density of water, kg/m^3

6.2.4 Camber

6.2.4.1 Camber of beams, trusses or girders, if required, shall be called for on the design drawings. Generally trusses and crane girders of 25 000 mm or greater span should be cambered for approximately the dead-plus-half-live-load deflection (see also Clause 17 for requirements for open-web joists).

6.2.4.2 Any special camber requirements necessary to bring a loaded member into proper relation with the work of other trades shall be stipulated on the design drawings.

6.2.5 Dynamic Effects

6.2.5.1 Suitable provision shall be made in the design for the effect of live load which induces impact or vibration, or both. In severe cases, such as structural supports for heavy machinery which causes substantial impact or vibration when in operation, the possibility of harmonic resonance, fatigue, or unacceptable vibration shall be investigated.

6.2.5.2 Special consideration shall be given to floor systems susceptible to vibration, such as large open floor areas free of partitions, to ensure that such vibration is acceptable for the intended use and occupancy. (A guide on floor vibrations is contained in Appendix F of this Standard.)

6.2.5.3 Unusually flexible structures (generally those whose ratio of height to effectively resisting width exceeds 4:1) shall be investigated for lateral vibrations under dynamic wind load. Lateral accelerations of the structure shall be checked to ensure that such accelerations are acceptable for the intended use and occupancy. (Information on lateral accelerations under dynamic wind loads can be found in Appendix G.)

6.2.6 Resistance to Fatigue

Structural steelwork shall be designed to resist the effects of fatigue under the specified loads in accordance with Clause 15.

6.2.7 Prevention of Permanent Deformation

6.2.7.1 For composite beams unshored during construction, the stresses in the tension flange of the steel beam due to the loads applied before the concrete strength reaches $0.75f'_c$ plus the stresses at the same location due to the remaining specified loads considered to act on the composite section shall not exceed $0.90F_y$.

6.2.7.2 Slip-resistant (friction) joints, in which the design load is assumed to be transferred by the slip resistance of the clamped faying surfaces, shall be proportioned, using the provisions of Clause 13.12, to resist without slipping, the moments and forces induced by the specified loads (see Clause 23).

6.3 Requirements Under Factored Loads

6.3.1 Strength

Structural steelwork shall be proportioned to resist moments and forces resulting from application of the factored loads acting in the most critical combination, taking into account the importance of the building, as specified in Clause 7 and taking into account the resistance factors as specified in the appropriate clauses of this Standard.

6.3.2 Overturning

The building or structure shall be designed to resist overturning resulting from application of the factored loads acting in the most critical combination, taking into account the importance of the building as specified in Clause 7, and taking into account the resistance factors as specified in the appropriate clauses of this Standard (see Clause 7).

6.3.3 Resistance to Earthquakes

6.3.3.1 In areas of known seismic activity buildings shall be designed to resist moderate earthquakes without significant damage and resist major earthquakes without collapse. Collapse is defined as the state which exists when exit of the occupants from the building has become impossible because of failure of the primary structure. The intent is to provide buildings with resistance to earthquake ground motion but not to slides, subsidence or active faulting in the immediate vicinity of the structure.

6.3.3.2 Unusual structures, highly irregular buildings and special-purpose industrial structures such as nuclear reactors, power plants and stacks should be treated as special problems with special design criteria in each instance, including possibly a dynamic analysis.

6.3.3.3 Structures designed to be resistant to earthquakes shall meet the requirements for structures analysed plastically as given in Clause 8.5 and in addition:

(a) the total drift per storey under the most severe design earthquake shall not exceed 0.03 of the storey height;

(b) beam webs yielding under cyclic loading, shall be designed so that local buckling does not occur.

6.3.3.4 In determining the lateral forces to be used in the design against earthquakes the type of construction, damping, ductility, and energy absorptive capacity of the structure shall be taken into consideration.*

* Information on numerical coefficients reflecting this behaviour is available in the National Building Code of Canada 1985.

6.4 Other Requirements

6.4.1 Expansion and Contraction

Suitable provision shall be made for expansion and contraction commensurate with the service and erection conditions of the structure.

6.4.2 Corrosion Protection

6.4.2.1 Steelwork shall have sufficient corrosion protection to minimize any corrosion likely to occur in the service environment.

6.4.2.2 Corrosion protection shall be provided by means of suitable alloying elements in the steel, by protective coatings or by other effective means, either singly or in combination.

6.4.2.3 Localized corrosion likely to occur from entrapped water, excessive condensation, or from other factors shall be minimized by suitable design and detail. Where necessary, positive means of drainage shall be provided.

6.4.2.4 If the corrosion protection specified for steelwork exposed to the weather, or to other environments in which progressive corrosion can occur, is likely to require maintenance or renewal during the service life of the structure, the steelwork so protected (exclusive of fill plates and shims) shall have a minimum thickness of 4.5 mm.

6.4.2.5 The minimum required thickness of steelwork situated in a non-corrosive environment and therefore not requiring corrosion protection is governed by the provisions of Clause 11.

6.4.2.6 Interiors of buildings conditioned for human comfort may be generally assumed to be non-corrosive environments; however, the need for corrosion protection shall be assessed and protection shall be furnished in those where it is deemed to be necessary.

6.4.2.7 Corrosion protection of the inside surfaces of enclosed spaces permanently sealed from any external source of oxygen is unnecessary.

7. Loads and Safety Criterion

7.1 Specified Loads

7.1.1 Except as provided for in Clauses 7.1.2 and 7.1.3, the following loads and influences as specified by the Regulatory Authority shall be considered in the design of a building and its structural steelwork:

D - Dead loads, including the mass of steelwork and all permanent materials of construction, partitions and stationary equipment multiplied by the acceleration due to gravity to convert mass to force, and the forces due to prestressing;

L - Live loads, including loads due to intended use and occupancy of the building, movable equipment, rain, soil or hydrostatic pressure, impact, and any other live loads stipulated by the applicable building by-law or the Regulatory Authority;

Q - Wind or earthquake loads;

T - Influences resulting from temperature changes, shrinkage or creep of component materials, or from differential settlement.

7.1.2 Where a building or a structural member can be expected to be subjected to loads, forces or other effects not listed in Clause 7.1.1 such effects shall be taken into account in the design based on the most appropriate information available.

7.1.3 If it can be shown by engineering principles, or if it is known from experience, that neglect of some or all of the effects due to temperature changes, shrinkage or creep of componenet materials or from differential settlement does not affect the structural safety or serviceability, they need not be considered in the calculations.

7.1.4 Suitable provision shall be made for loads imposed on the steel structure during its erection. During subsequent construction of the building, suitable provision shall be made to support the construction loads on the steel structure with an adequate margin of safety.

7.2 Safety Criterion and Effect of Factored Loads

7.2.1 Safety Criterion

A building and its structural steelwork shall be designed to have sufficient strength or stability, or both, such that:

Factored Resistance > Effect of Factored Loads

where Factored Resistance is determined in accordance with other clauses of this Standard and Effect of Factored Load is determined in accordance with Clauses 7.2.2 through 7.2.5. In cases of overturning, uplift and stress reversal, no positive anchorage is required if the stabilizing effect of dead load multiplied by a load factor less than one given in Clause 7.2.3 is greater than the effect of loads tending to cause overturning, uplift and stress reversal multiplied by load factors greater than one given in Clause 7.2.3.

7.2.2 Effect of factored loads, in force units, is the structural effect due to the specified loads multiplied by load factors α defined in Clause 7.2.3, a load combination factor ψ defined in Clause 7.2.4, and an importance factor γ defined in Clause 7.2.5. The factored load combinations shall be taken as follows:

$$\alpha_D D + \gamma\psi(\alpha_L L + \alpha_Q Q + \alpha_T T)$$

7.2.3 Load factors, α, shall be taken as follows:

α_D = 1.25, except that when the dead load resists overturning, uplift, or reversal of load effect, α_D = 0.85.

α_L = 1.50; α_Q = 1.50; α_T = 1.25.

7.2.4 The load combination factor, ψ, shall be taken as follows:

(a) When only one of L, Q and T act, ψ = 1.00;

(b) When two of L, Q and T act, $\psi = 0.70$;

(c) When all of L, Q and T act, $\psi = 0.60$.

The most unfavourable effect shall be determined by considering L, Q and T acting alone with $\psi = 1.00$ or in combination with $\psi = 0.70$ or 0.60.

7.2.5 The Importance Factor, γ, shall be not less than 1.00 for all buildings, except that for buildings where it can be shown that collapse is not likely to cause injury or other serious consequences, it shall be not less than 0.8.

8. Analysis of Structure

8.1 General

8.1.1 In proportioning the structure to meet the various design requirements of Clause 6, the methods of analysis given in this Clause shall be used. The distribution of internal forces and bending moments shall be determined, both under the specified loads to satisfy the requirements of serviceability and fatigue in Clause 6 and under the factored loads as required to satisfy strength and overturning requirements in Clause 7.

8.1.2 Two basic types of construction and associated design assumptions, designated "Continuous" and "Simple" are permitted for all or part of a structure under this Standard. The distribution of internal forces and bending moments throughout the structure will depend on the type or types of construction chosen and the forces to be resisted.

8.2 Continuous Construction

In continuous construction, the beams, girders and trusses are rigidly framed, or are continuous over supports. Connections are generally designed to resist the bending moments and internal forces computed by assuming that the original angles between intersecting members remain unchanged as the structure is loaded.

8.3 Simple Construction

8.3.1 Simple construction assumes that the ends of beams, girders and trusses are free to rotate under load in the plane of loading. Resistance to lateral loads, including sway effects, shall be ensured by a suitable system of bracing or shear walls or by the design of part of the structure as continuous construction, except as provided in Clause 8.3.2.

8.3.2 A building frame designed to support gravity loads on the basis of simple construction may be proportioned to resist lateral loads due to wind or earthquake by distributing the moments resulting from such loading among selected joints of a frame by a recognized empirical method provided that:

(a) The connection and connected members are proportioned to resist the moments and forces caused by lateral loads;

(b) The connected members have solid webs;

(c) The beam or girder can support the full gravity load when assumed to act as a simple beam;

(d) The connection has adequate capacity for inelastic rotation when subjected to full gravity and lateral loads;

(e) The mechanical fasteners or welds of the connection are proportioned to resist 1.5 times the moments and forces produced by the factored gravity and lateral loads; and

(f) In assessing the stability of the structure, the flexibility of the connection is considered.

8.4 **Elastic Analysis**
Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by an analysis which assumes that individual members behave elastically.

8.5 **Plastic Analysis**
Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by a plastic analysis provided that:

(a) The steel used has $F_y < 0.80F_u$ and exhibits the load-strain characteristics necessary to achieve moment redistribution;

(b) The width-thickness ratios meet the requirements of Class 1 sections as given in Clause 11.2;

(c) The members are braced laterally in accordance with the requirements of Clause 13.7;

(d) Web stiffeners are supplied on a member at a point of load application or where a plastic hinge would form;

(e) Splices in beams or columns are designed to transmit 1.1 times the maximum computed moment under factored loads at the splice location or $0.25M_p$, whichever is greater.

(f) Members are not subject to repeated heavy impact or fatigue;

(g) The influence of inelastic deformation on the strength of the structure shall be taken into account. (See also Clause 8.6.)

8.6 Stability Effects

8.6.1 The analyses referred to in Clauses 8.4 and 8.5 shall include the sway effects produced by the vertical loads acting on the structure in its displaced configuration, unless the structure is designed in accordance with the provisions of Clause 8.6.3.

For certain types of structures where the vertical loads are small, where the structure is relatively stiff and where the lateral load resisting elements are well distributed, the sway effects may not have a significant influence on the design of the structure (see Clause 9.3.2(b)).

8.6.2 For structures in which the sway effects have been included in the analysis to determine the design moments and forces (see Appendix I) the effective length factors for members shall be based on the side-sway prevented condition (see Clause 9.3.2(a)) and,

(a) Where a loading combination produces significant relative lateral displacements of the column ends, the sway effects shall include the effect of the vertical loads acting on the displaced structure;

(b) However, in no case shall the sway effects be taken as less than those calculated by assuming that the vertical loads act on the structure assumed to be displaced by an amount equal to the maximum out-of-plumbness consistent with the erection tolerances specified in Clause 29.7.1;

(c) A deflected configuration in which the erection tolerances are opposite in sense in adjacent storeys, may produce sway effects which govern the design of beam-to-column connections, diaphragms and other elements.

8.6.3 For structures in which the sway effects have not been included in the analysis, the use of effective length factors greater than 1.0 (side-sway permitted case) for the design of columns, provides an approximate method of accounting for the sway effects in moment resisting frames (see Clause 9.3.3). This provision shall not be used for structures analysed in accordance with Clause 8.5.

9. Design Lengths of Members

9.1 Simple Span Flexural Members

Beams, girders, and trusses may be designed on the basis of simple spans whose length may be taken as the distance between centres of gravity of supporting members. Alternatively, the span length of beams and girders may be taken as the actual length of such members measured centre-to-centre of end connections. The length of trusses designed as simple spans may be taken as the distance between the extreme working points of the system of triangulation employed. In all cases the design of columns or other supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder, or truss may actually be connected or supported.

9.2 Continuous Span Flexural Members

Beams, girders, or trusses having full or partial end restraint due to continuity or cantilever action, shall be proportioned to carry all moments, shears, and other forces at any section assuming the span, in general, to be the distance between centres of gravity of supporting members. Supporting members shall be proportioned to carry all moments, shears, and other forces induced by the continuity of the supported beam, girder, or truss.

9.3 Compression Members

9.3.1 Compression members shall be designed on the basis of their effective length (KL), the product of effective length factor (K), and unbraced length (L). Unless otherwise specified in this Standard the unbraced length (L) shall be taken as the length of the compression member centre-to-centre of restraining members. The unbraced length may differ for different cross-sectional axes of the compression members. At the bottom storey of a multistorey structure, or for a single-storey structure, (L) shall be taken as the length from the top of the base plate to the centre of restraining members at the next higher level.

9.3.2 The effective length factor (K) shall be taken as 1.0 for compression members of frames:

(a) In which sway effects (see Clause 8.6.2) have been included in the analysis used to determine the design moments and forces; or

(b) In which the sway effects in addition to the lateral loads are resisted by bracing or shear walls,

unless the degree of rotational restraint afforded at the ends of the unbraced lengths shows that a value of K less than 1.0 is applicable. (For recommended values of K and a method of computing K, based on rotational restraint, see Appendices B and C respectively, side-sway prevented case.)

9.3.3 For structures with moment resisting frames in which sway effects have not been included in the analysis used to determine the design moments and forces (see Clause 8.6.3), the effective length factor shall be determined from the degree of rotational and translational restraint afforded at the ends of the unbraced length, but shall be not less than 1. (For recommended values of K and a method of computing K, see Appendices B and C respectively, side-sway permitted case.)

9.3.4 **Compression Members in Trusses**

Unless otherwise specified in this Standard or unless analysis shows that a smaller value is applicable, the effective length factor (K) shall be taken as 1.0 for compression members in trusses.

10. Slenderness Ratios

10.1 **General**

The slenderness ratio of a compression member shall be taken as the ratio of effective length (KL) to the corresponding radius of gyration (r). The slenderness ratio of a tension member shall be taken as the ratio of unbraced length (L) to the corresponding radius of gyration.

10.2 **Maximum Slenderness Ratio**

10.2.1 The slenderness ratio of a compression member shall not exceed 200.

10.2.2 The slenderness ratio of a tension member shall not exceed 300. This limit may be waived if other means are provided to control flexibility, sag, vibration, and slack in a manner commensurate with the service conditions of the structure, or if it can be shown that such factors are not detrimental to the performance of the structure or of the assembly of which the member is a part.

11. Width-Thickness Ratios: Compression Elements

11.1 Classification of Sections

11.1.1 For the purposes of this Standard, structural sections shall be designated as Class 1, 2, 3 or 4 depending on the maximum width-thickness ratios of their elements subject to compression, and as otherwise specified in Clause 11.1.3.

11.1.2 **Class 1** sections (plastic design sections) will permit attainment of the plastic moment and subsequent redistribution of bending moment.

Class 2 sections (compact sections) will permit attainment of the plastic moment but need not allow for subsequent moment redistribution.

Class 3 sections (non-compact sections) will permit attainment of the yield moment.

Class 4 sections will generally have local buckling of elements in compression as the limit state of structural capacity.

11.1.3 Class 1 sections shall, when subject to flexure, have an axis of symmetry in the plane of loading and shall, when subject to axial compression, be doubly symmetric. Class 2 sections shall, when subject to flexure, have an axis of symmetry in the plane of loading unless the effects of asymmetry of the section are included in the analysis.

11.2 Width and Thickness

11.2.1 For elements supported along one edge only, parallel to the direction of compressive force, the width shall be taken as follows:

(a) For plates, the width (b) is the distance from the free edge to the first row of fasteners or line of welds;

(b) For legs of angles, flanges of channels and zees, and stems of tees, the width (b) is the full nominal dimension;

(c) For flanges of beams and tees, the width (b) is one-half the full nominal dimension.

11.2.2 For elements supported along two edges parallel to the direction of compressive force the width shall be taken as follows:

(a) For flange or diaphragm plates in built-up sections the width (b) is the distance between adjacent lines of fasteners or lines of welds;

(b) For flanges of rectangular hollow structural sections the width (b) is the clear distance between webs less the inside corner radius on each side;

(c) For webs of built-up sections the width (h) is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used;

(d) For webs of hot rolled sections the width (h) is the clear distance between flanges.

11.2.3 The thickness of elements is the nominal thickness. For tapered flanges of rolled sections, the thickness is the nominal thickness halfway between a free edge and the corresponding face of the web.

11.3 **Maximum Width-Thickness Ratios of Elements Subject to Compression**

The width-thickness ratio of elements subject to compression shall not exceed the limits given in Table 1 for the specified section classification.

12. Gross and Net Areas

12.1 **Application**

In general, tension members shall be proportioned on the basis of net area and compression members on the basis of gross area. (For beams and girders see Clause 16.)

12.2 **Gross Area**

Gross area shall be computed by summing the products of the thickness and gross width of each element, as measured normal to the axis of the member.

12.3 **Net Area**

12.3.1 Net area shall be computed by summing the products of the thickness and the net width of each element, as measured normal to the axis of the member. Net width and area of parts containing holes shall be

Table 1
Width-Thickness Ratios: Compression Elements

	Section Classification			
Description of Element	**Class 1 Plastic Design**	**Class 2 Compact**	**Class 3 Non-compact**	**Class 4* Slender**
Legs of angles and elements supported along one edge, except as noted	--	--	$\frac{b}{t} \leq \frac{200}{\sqrt{F_y}}$	See Clause 13
Angles in continuous contact with other elements; plate girder stiffeners	--	--	$\frac{b}{t} \leq \frac{200}{\sqrt{F_y}}$	See Clause 13
Stems of T sections	$\frac{b}{t} \leq \frac{145\dagger}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{170\dagger}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{340}{\sqrt{F_y}}$	See Clause 13
Flanges of I or T sections; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact‡	$\frac{b}{t} \leq \frac{145}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{170}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{200}{\sqrt{F_y}}$	See Clause 13
Flanges of channels	--	--	$\frac{b}{t} \leq \frac{200}{\sqrt{F_y}}$	See Clause 13
Flanges of rectangular hollow structural sections	$\frac{b}{t} \leq \frac{420}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{670}{\sqrt{F_y}}$	See Clause 13

Table 1 (continued)

	Section Classification			
Description of Element	**Class 1 Plastic Design**	**Class 2 Compact**	**Class 3 Non-compact**	**Class 4* Slender**
Flanges of box sections, flange cover plates and diaphragm plates, between lines of fasteners or welds	$\frac{b}{t} \leqslant \frac{525}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{525}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{670}{\sqrt{F_y}}$	See Clause 13
Perforated cover plates	--	--	$\frac{b}{t} \leqslant \frac{840}{\sqrt{F_y}}$	—
Webs in axial compression	$\frac{h}{w} \leqslant \frac{670}{\sqrt{F_y}}$	$\frac{h}{w} \leqslant \frac{670}{\sqrt{F_y}}$	$\frac{h}{w} \leqslant \frac{670}{\sqrt{F_y}}$	See Clause 13
Webs in flexural compression	$\frac{h}{w} \leqslant \frac{1100}{\sqrt{F_y}}$	$\frac{h}{w} \leqslant \frac{1370}{\sqrt{F_y}}$	$\frac{h}{w} \leqslant \frac{1810}{\sqrt{F_y}}$	See Clause 13

Table 1 (continued)

Description of Element	Section Classification: Class 1 Plastic Design	Class 2 Compact	Class 3 Non-compact	Class 4* Slender
Webs in combined flexural and axial compression	$\frac{h}{w} \leq \frac{1100}{\sqrt{F_y}}\left(1 - 1.40\frac{C_f}{C_y}\right)$ §	when $\frac{C_f}{C_y} \leq 0.15$, $\frac{h}{w} \leq \frac{1370}{\sqrt{F_y}}\left(1 - 1.28\frac{C_f}{C_y}\right)$ when $\frac{C_f}{C_y} > 0.15$, $\frac{h}{w} \leq \frac{1180}{\sqrt{F_y}}\left(1 - 0.43\frac{C_f}{C_y}\right)$	when $\frac{C_f}{C_y} \leq 0.15$, $\frac{h}{w} \leq \frac{1810}{\sqrt{F_y}}\left(1 - 1.69\frac{C_f}{C_y}\right)$ when $\frac{C_f}{C_y} > 0.15$, $\frac{h}{w} \leq \frac{1470}{\sqrt{F_y}}\left(1 - 0.54\frac{C_f}{C_y}\right)$	See Clause 13
Circular hollow sections	$\frac{D}{t} \leq \frac{13\ 000}{F_y}$	$\frac{D}{t} \leq \frac{18\ 000}{F_y}$	$\frac{D}{t} \leq \frac{23\ 000}{F_y}$	--

* Class 4 includes all sections not otherwise specified.

† See Clause 11.1.3

‡ Can be considered as Class 1 or Class 2 sections if angles are continuously connected by adequate mechanical fasteners or welds and there is an axis of symmetry in the plane of loading.

§ $\frac{h}{w}$ need not be less than $\frac{670}{\sqrt{F_y}}$

computed in accordance with Clause 12.3.3.

12.3.2 **Dimensions of Holes**
In computing net area the width of the bolt holes normal to the axis of the member shall be assumed 2 mm larger than the hole dimension specified.

12.3.3 For a series of holes extending across a part of any diagonal or zigzag line, the net width of the part shall be computed by deducting from the gross width the sum of all hole widths in the series and adding for each gauge distance (g) in the series the quantity:

$$\frac{s^2}{4g}$$

where

s = longitudinal spacing (pitch) in millimetres of any two successive holes

g = transverse spacing (gauge) in millimetres of the same two holes

12.3.4 The critical net area of the part is obtained from that series of holes which gives the least net width; however, the net area through one or more holes shall not be taken as greater than the following limits:

(a) $0.85A_g$ when $F_y/F_u \leq 0.75$

(b) $0.90A_g$ when $0.75 < F_y/F_u \leq 0.85$

(c) $0.95A_g$ when $0.85 < F_y/F_u$

12.3.5 For angles the gross width shall be the sum of the widths of the legs minus the thickness. The gauge for holes in opposite legs shall be the sum of the gauges from the heel of the angle minus the thickness.

12.3.6 In computing the net area across plug or slot welds the weld metal shall not be taken as adding to the net area.

12.4 **Pin-Connected Tension Members**

12.4.1 In pin-connected tension members, the net area across the pin hole, normal to the axis of the member, shall be at least 1.33 times the cross-sectional area of the body of the member. The net area of any section on either side of the axis of the member measured at an angle of 45° or less to the axis of the member, shall not be less than 0.9 times the cross-sectional area of the body of the member.

12.4.2 The distance from the edge of the pin hole to the edge of the member, measured transverse to the axis of the member, shall not exceed 4 times the thickness of the material at the pin hole.

12.4.3 The diameter of a pin hole shall be not more than 1 mm larger than the diameter of the pin.

13. Member and Connection Resistance

13.1 General

To meet the strength requirements of this Standard, all factored resistances, as determined in this Clause, shall be greater than or equal to the effect of factored loads determined in accordance with Clause 7.2 and ϕ shall be taken as 0.90 unless otherwise specified.

13.2 Axial Tension

The factored tensile resistance, T_r, developed by a member subjected to an axial tension force shall be taken as:

(a) The lesser of

(i) $T_r = \phi A_n F_y$ when $A_n/A_g > F_y/F_u$

$= \phi\left(F_u \dfrac{A_n}{A_g}\right) A_n$ when $A_n/A_g < F_y/F_u$

(ii) $T_r = 0.85\phi A_n F_u$

(b) On net area at pin connections

$$T_r = 0.75\phi A_n F_y$$

13.3 Axial Compression*

13.3.1 The factored axial compressive resistance for W shapes and for hollow structural sections manufactured according to CSA Standard CAN3-G40.20-M, Class C (cold formed non-stress relieved) and conforming to the requirements of Clause 11 of this Standard for Class 1, 2 or 3 sections, shall be taken as*:

(a) $0 < \lambda < 0.15$, $C_r = \phi A F_y$

(b) $0.15 < \lambda < 1.0$, $C_r = \phi A F_y(1.035 - 0.202\lambda - 0.222\lambda^2)$

(c) $1.0 < \lambda \leq 2.0$, $C_r = \phi AF_y(-0.111 + 0.636\,\lambda^{-1} + 0.087\,\lambda^{-2})$

(d) $2.0 < \lambda \leq 3.6$, $C_r = \phi AF_y(0.009 + 0.877\,\lambda^{-2})$

(e) $3.6 < \lambda$, $C_r = \phi AF_y\,\lambda^{-2} = \phi A\left[\frac{1\,970\,000}{(KL/r)^2}\right]$

where $\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}$

Values of $\sqrt{\frac{F_y}{\pi^2 E}}$ and $\frac{F_y}{\pi^2 E}$ to compute λ and λ^2 respectively are given in Table 2.

*These expressions are applicable to W shapes up to 610 mm deep and can be assumed to be valid for other doubly symmetric Class 1, 2 or 3 sections, except for solid round non-stress relieved cold straightened bars greater than 50 mm diameter (refer to CSA Standard S37-M, Antenna Towers and Antenna Supporting Structures). Welded H-shapes should have flange edges flame cut. Singly symmetric, asymmetric or cruciform sections should be checked as to whether torsional-flexural buckling is critical. Curves similar to those described by the equations in Clause 13.3.1 but taking into account differences in residual stress patterns, shapes, etc., for fully stress relieved sections, hollow structural sections and jumbo sections are given in "Guide to Stability Design Criteria for Metal Compression Members", 3rd Edition, published by the Structural Stability Research Council.

Table 2

Values of $\sqrt{\frac{F_y}{\pi^2 E}}$ and $\frac{F_y}{\pi^2 E}$

F_y	$\sqrt{\frac{F_y}{\pi^2 E}}$	$\frac{F_y}{\pi^2 E}$
230	0.0108	0.000 116
260	0.0115	0.000 132
300	0.0123	0.000 152
350	0.0133	0.000 177
380	0.0139	0.000 192
400	0.0142	0.000 203
480	0.0156	0.000 243
700	0.0188	0.000 355

For unstiffened webs, $a/h = \infty$ and $\tau = 1$, $\eta = 0$, $k_v = 5.34$.

The values given in Table 3 may be used. The gross area of a web shall be taken as the product of the web depth (h) and the web thickness (w), except that for rolled shapes the overall depth (d) may be substituted for h.

13.4.2 **Plastic Analysis**
In structures designed on the basis of a plastic analysis, as defined in Clause 8.5 the factored shear resistance, V_r, developed by the web of a flexural member subjected to shear shall be taken as:

$$V_r = 0.55\phi w d F_y$$

Table 3
Coefficients for Shear Formulae

a/h	τ	η	k_v
0.25	0.160	0.485	89.4
0.33	0.178	0.475	52.2
0.50	0.225	0.447	25.4
0.67	0.280	0.415	16.0
0.75	0.307	0.400	13.5
1.00	0.388	0.354	9.34
1.25	0.459	0.312	7.90
1.50	0.520	0.277	7.12
1.75	0.570	0.248	6.65
2.00	0.613	0.224	6.34
2.25	0.648	0.203	6.13
2.50	0.678	0.186	5.98
2.75	0.704	0.171	5.87
3.00	0.726	0.158	5.78
Infinity	1.000	0	5.34

13.4.3 **Maximum Slenderness**
The slenderness ratio (h/w) of a web shall not exceed:

$$83\,000/F_y$$

where

F_y = specified minumum yield point of the compression flange steel (see Clause 16.3).

This limit may be waived if analysis indicates that compression flange buckling into the web will not occur at factored load.

13.4.4 **Gusset Plates**

The total factored shear resistance of the gross area of gusset plates shall be taken as:

$$V_r = 0.50\phi A_g F_y$$

13.4.5 **Pins**

The total factored shear resistance of the nominal area of pins shall be taken as:

$$V_r = 0.66\phi A F_y$$

13.4.6 **Combined Shear and Moment in Girders**

Transversely stiffened girders depending on tension-field action to carry shear, with $h/w > \sqrt{k_v/F_y}$, shall be proportioned in such a way that the following limits are observed:

$$\frac{V_f}{V_r} \leqslant 1.0$$

$$\frac{M_f}{M_r} \leqslant 1.0$$

$$0.727 \frac{M_f}{M_r} + 0.455 \frac{V_f}{V_r} \leqslant 1.0$$

where

V_r is established according to Clause 13.4 and M_r is established according to Clause 13.5.1 and Clause 13.5.2 as applicable.

13.5 Bending

13.5.1 **Laterally Supported Members**

The factored moment resistance, M_r, developed by a member subjected to bending moments, where continuous lateral support is provided to the compression flange shall be taken as:

(a) For Class 1 and Class 2 sections:

$$M_r = \phi Z F_y = \phi M_p$$

(b) For Class 3 sections:

$$M_r = \phi S F_y = \phi M_y$$

(c) For Class 4 sections:

(i) When both the web and the compression flange fall within Class 4 of Table 1, the value M_r shall be determined in accordance with Clause 14. The calculated value, F_y, applicable to cold formed members, shall be determined by using only the values for F_y and F_u that are specified in the relevant structural steel material standard;

(ii) For beams or girders whose flanges meet the requirements of Class 3 and whose webs exceed the limits for Class 3, see Clause 16;

(iii) For beams or girders whose webs meet the requirements of Class 3 and whose flanges exceed the limits for Class 3;

$$M_r = \phi S F_{cr}$$

where

F_{cr} = the critical stress corresponding to local buckling of the compression elements determined in accordance with Clause 14.

Alternatively, the moment resistance may be calculated as

$$M_r = \phi S_e F_y$$

where

S_e = the effective section modulus determined using an effective flange width of $670t/\sqrt{F_y}$ for flanges supported along two edges parallel to the direction of stress or an effective width of $200t/\sqrt{F_y}$ for flanges supported along one edge parallel to the direction of stress. In no case, for flanges supported along one edge, shall b/t exceed 60.

13.5.2 Laterally Unsupported Members

Where continuous lateral support is not provided to the compression flange of a member subjected to bending, the factored moment resistance, M_r, may be taken as:

(a) For Doubly Symmetric Class 1 and Class 2

Sections:

(i) When $M_u > \frac{2}{3} M_p$

$$M_r = 1.15\phi M_p \left[1 - \frac{0.28M_p}{M_u}\right]$$ but not greater than ϕM_p

(ii) When $M_u \leqslant \frac{2}{3} M_p$

$$M_r = \phi M_u$$

where

$$M_u = \frac{\pi}{\omega L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

For I-shaped members bent about the major axis, M_u, may be taken as:

$$M_u = \frac{S}{\omega} \sqrt{\sigma_1^{\,2} + \sigma_2^{\,2}}$$

$$\sigma_1 = \frac{140\ 000}{Ld/A_f}$$

$$\sigma_2 = \frac{1\ 700\ 000}{(L/r_t)^2}$$

where

ω = $0.6 + 0.4M_{f1}/M_{f2}$ for members bent in single curvature

ω = $0.6 - 0.4M_{f1}/M_{f2}$ for members bent in double curvature, but not less than 0.4

L = unsupported length of compression flange in millimetres in which M_{f1} is the smaller and M_{f2} the larger bending moment at the ends of the unsupported length, taken about the strong axis of the member

ω = 1.0 when the bending moment at any point within the unsupported length is larger than M_{f2} or when there is no effective lateral support for the compression flange at one of the ends of the unsupported length

C_w = 0 for hollow structural sections.

(b) For Doubly Symmetric Class 3 and Class 4 Sections and for Channels Prevented from Twisting:

(i) When $M_u > \frac{2}{3}M_y$

$$M_r = 1.15\phi M_y \left(1 - \frac{0.28M_y}{M_u}\right)$$

but not greater than ϕM_y for Class 3 sections and ϕSF_{cr} for Class 4 sections:

(ii) When $M_u < \frac{2}{3}M_y$

$$M_r = \phi M_u$$

where

$$M_u = \frac{\pi}{\omega L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

For I-shaped members and channels M_u may be taken as:

$$M_u = \frac{S}{\omega} \sqrt{\sigma_1^{\,2} + \sigma_2^{\,2}}$$

where

σ_1 and σ_2 are as defined in Clause 13.5.2(a)

$\sigma_2 = 0$ for channels

(c) For unsymmetric shapes a rational method of analysis such as given in the Structural Stability Research Council's "Guide to Stability Design Criteria for Metal Structures" should be used.

13.6 Lateral Bracing for Members in Structures Analysed Plastically

Members in structures or portions of structures in which the distributions of moments and forces have been determined by a plastic analysis shall be braced to resist lateral and torsional displacement at all hinge locations. The laterally unsupported distance, L_{cr}, from such braced hinge locations to the nearest adjacent point on the frame similarly braced shall not exceed:

$$L_{cr} = 550 r_y / \sqrt{F_y} \quad \text{for } \frac{M_{f1}}{M_{f2}} > 0.5$$

$$L_{cr} = 980 r_y / \sqrt{F_y} \quad \text{for } \frac{M_{f1}}{M_{f2}} < 0.5$$

where

$\frac{M_{f1}}{M_{f2}}$ is equal to the ratio of the smaller moment to the larger moment at opposite ends of the unbraced length, in the plane of bending considered; positive when the member is bent in single curvature and negative when bent in double curvature and

$$-1.0 < \frac{M_{f1}}{M_{f2}} < 1.0$$

Both bracing requirements should be checked and the more severe shall govern the location of the braced point. Bracing is not required at the location of the last hinge to form in the failure mechanism assumed as the basis for proportioning the structure. Except for the regions specified above, the maximum unsupported length of members in structures analysed plastically need be not less than that which would be permitted for the same members in structures analysed elastically.

13.7 Axial Compression and Bending*

13.7.1 Member Strength and Stability - Class 1 and Class 2 Sections

Members required to resist bending moments and an axial compressive force shall be proportioned so that:

(a) In a state of complete yielding (partly in tension, partly in compression) the section is capable of holding in equilibrium the factored moments and axial load. Conservatively, this requirement is satisfied if the section meets the requirements of Clause 13.8.3(a);

(b) $$\frac{C_f}{C_r} + \frac{\omega_x M_{fx}}{M_{rx}\left[1 - \frac{C_f}{C_{ex}}\right]} + \frac{\omega_y M_{fy}}{M_{ry}\left[1 - \frac{C_f}{C_{ey}}\right]} < 1.0$$

where

M_{rx} is defined in Clause 13.5.2(a) and M_{ry} is defined in Clause 13.5.1(a).

* More detailed methods of determining the resistance of columns subject to biaxial bending are given in Appendix L.

13.7.2 Member Strength and Stability - Class 1 and Class 2 Sections of I-Shaped Members

Members required to resist bending moments and an axial compressive force shall be proportioned so that:

(a) $$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} < 1.0$$

where

M_r is defined in Clause 13.5.1(a)

(b) $$\frac{C_f}{C_r} + \frac{0.85M_{fx}}{M_{rx}} + \frac{0.60M_{fy}}{M_{ry}} \leq 1.0$$

where

$C_r = \phi A F_y$

M_r is defined in Clause 13.5.1(a)

(c) $$\frac{C_f}{C_r} + \frac{\omega_x M_{fx}}{M_{rx}\left[1 - \frac{C_f}{C_{ex}}\right]} + \frac{\omega_y M_{fy}}{M_{ry}\left[1 - \frac{C_f}{C_{ey}}\right]} \leq 1.0$$

where

C_r is defined in Clause 13.3.1
M_{rx} is defined in Clause 13.5.2(a)
M_{ry} is defined in Clause 13.5.1(a)

13.7.3 **Member Strength and Stability - Class 3 and Class 4 Sections**

Members required to resist bending moments and an axial compressive force shall be proportioned so that:

(a) $$\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$$

where

$C_r = \phi A F_y$

M_r is defined in Clause 13.5.1(b) or 13.5.1(c)(i) and (c)(iii)

(b) $$\frac{C_f}{C_r} + \frac{\omega_x M_{fx}}{M_{rx}\left[1 - \frac{C_f}{C_{ex}}\right]} + \frac{\omega_y M_{fy}}{M_{ry}\left[1 - \frac{C_f}{C_{ey}}\right]} \leq 1.0$$

where

C_r is defined in Clause 13.3
M_{rx} is defined in Clause 13.5.2(b)
M_{ry} is defined in Clause 13.5.1(b) or 13.5.1(c)(i) and (c)(iii)

13.7.4 **Values of ω**

Unless otherwise determined by analysis, the following values shall be used for ω:

(a) Members not subjected to transverse loads between supports:

(i) For members of frames analysed in accordance with Clause 8.6.2,
$\omega = 0.6 + 0.4M_{f1}/M_{f2}$ for members bent in single curvature
$\omega = 0.6 - 0.4M_{f1}/M_{f2}$ for members bent in double curvature, but not less than 0.4

where

M_{f1}/M_{f2} = ratio of the smaller moment to the larger moment at opposite ends of the unbraced length, in the plane of bending considered;

(ii) For members of frames analysed in accordance with Clause 8.6.3,

$\omega = 0.85$ for members bent in double curvature or subject to moment at one end.
$\omega = 1.0$ for members bent in single curvature due to moments at both ends

(b) Members subjected to distributed load or series of point loads between supports: $\omega = 1.0$

(c) Members subjected to a comcentrated load or moment between supports: $\omega = 0.85$.

For the purpose of design, members subjected to concentrated load (or moment) between supports (e.g., crane columns) may be considered to be divided into two segments at the point of load (or moment) application. Each segment shall then be treated as a member which depends on its own flexural stiffness to prevent side-sway in the plane of bending considered and ω shall be taken as 0.85. In computing the slenderness ratio KL/r, for use in Clause 13.7, the total length of the member shall be used.

13.8 Axial Tension and Bending

Members required to resist both bending moments and an axial tensile force shall be proportioned so that:

(a) $\dfrac{T_f}{T_r} + \dfrac{M_f}{M_r} < 1.0$

where

$M_r = \phi M_p$ for Class 1 and Class 2 sections
$M_r = \phi M_y$ for Class 3 and Class 4 sections

(b) $\dfrac{M_f}{M_r} - \dfrac{T_f Z}{M_r A} < 1.0$ for Class 1 and Class 2 sections

or

$\dfrac{M_f}{M_r} - \dfrac{T_f S}{M_r A} < 1.0$ for Class 3 and Class 4 sections

where

M_r is defined in Clause 13.5.1(c) or 13.5.2

13.9 **Load Bearing**

The factored bearing resistance, B_r, developed by a member or portion of a member, subjected to bearing shall be taken as:

(a) On the contact area of machined, accurately sawn or fitted parts $B_r = 1.50\phi F_y A$

(b) On expansion rollers or rockers

$B_r = 0.000\ 13\phi DLF_y^2$

where

B_r is in newtons

ϕ is taken as 0.90

D and L are the diameter and length respectively of roller or rocker

F_y is the specified minimum yield point of the weaker part in contact

(c) In bearing-type connections

$B_r = \phi tneF_u < 3\phi tdnF_u$

where

ϕ is taken as 0.67

F_u is the tensile strength of the plate

The ratio of end distance to bolt diameter shall meet the requirements of Clause 22.8.

13.10 Bolts in Bearing-Type Connections

13.10.1 General

For bolts in bearing-type connections ϕ shall be taken as 0.67 to ensure that the connection will not fail before the member.

13.10.2 Bolts in Shear

The factored resistance developed by a bolted joint subjected to shear shall be taken as the lesser of:

(a) The factored bearing resistance, B_r, given in Clause 13.9(c); or

(b) The factored shear resistance of the bolts taken as:

$$V_r = 0.60\phi nmA_bF_u{}^*$$

When the bolt threads are intercepted by any shear plane, the factored shear resistance of any joint shall be taken as 70 per cent of V_r.

For joints longer than 1300 mm the shearing resistance shall be taken as 80 per cent of the above values.

* For A325M bolts F_u is 830 MPa and for A490M bolts F_u is 1040 MPa. For A325 bolts 1 inch or less in diameter F_u is 825 MPa and for A325 bolts greater than 1 inch in diameter F_u is 725 MPa. For A490 bolts, F_u is 1035MPa.

ASTM Standards A325 and A490 are written in Imperial Units. Accordingly, bolt diameters are shown in the Imperial System.

13.10.3 Bolts in Tension

The factored tensile resistance developed by a bolted joint, T_r, subjected to tension, T_f, shall be taken as:

$$T_r = 0.75\phi nA_bF_u$$

13.10.4 Bolts in Combined Shear and Tension

A bolt is a joint required to develop resistance to both tension and shear shall be proportioned so that the following relationship is satisfied:

$$\frac{V_f^2}{m^2} + \beta T_f^2 < 0.56\phi^2\beta(A_bF_u)^2$$

where

β = an interaction factor derived from test results
= 0.69 for A325 bolts, shear plane through shank
= 0.41 for A325 bolts, shear plane through threads
= 0.56 for A490 bolts, shear plane through shank
= 0.30 for A490 bolts, shear plane through threads

Except that V_f shall not exceed V_r given in Clause 13.10.2

13.11 Bolts in Slip-Resistant (Friction-Type) Connections

13.11.1 General

The requirement for a slip-resistant connection is that under the forces and moments produced by specified loads, slip of the assembly shall not occur. In addition the effect of factored loads shall not exceed the resistances of the connection as given in Clause 13.10.

13.11.2 Shear Connections

The slip resistance, V_s, of a bolted joint, subjected to shear, V, shall be taken as:

$$V_s = 0.26 \mu m n A_b F_u$$

where μ, a function of slip probability, is expressed as a function of bolt type and condition of the faying surfaces of the parts. Representative values corresponding to a 5 per cent probability of slip are given in Table 4.

13.11.3 Connections in Combined Shear and Tension

Bolts in a joint required to develop resistance to both tension and shear shall be proportioned so that the following relationship is satisfied for the specified loads:

$$\frac{V}{V_s} + 1.9 \frac{T}{n A_b F_u} \leq 1.0$$

where

V_s = slip resistance as defined in Clause 13.11.2.

13.12 Welds

13.12.1 The factored resistance of welded joints is dependent upon the strength of the electrode used. Conditions wherein the electrode is "matched" to the base metal are given in Table 5(a).

13.12.2 The factored resistance of welded joints for the weld metal and base metal shall be as shown in Table 5(b).

Table 4
Values of μ

Category	Steels		Steel Surface Treatment*	Bolts	
	CSA G40.21-M	ASTM		A325M A325	A490M A490
1	Carbon and low-alloy steels except quenched and tempered		Tight clean mill scale		
2	300W, 300WT	A36	Vinyl wash primer	0.59	0.51
3	300W, 300WT	A36, A441	Blast-cleaned, organic zinc rich paint		
4			Blast-cleaned		
5			Blast-cleaned, inorganic zinc rich paint	0.99	0.87
6			Blast-cleaned, metallized		
7		A514	Blast-cleaned	0.69	0.60
8	300W, 300WT	A36, A441	Hot-dipped galvanized	0.31	0.27
9			Hot-dipped galvanized then wire-brushed or blasted	0.76	0.66

* See Clause 23.3.4 for steel surface treatment requirements.

13.12.3 The factored compressive resistance of joints utilizing partial joint penetration groove welds shall be based on the effective throat area of the welds plus the area of the base metal fitted in contact bearing.

Note: Compression joints that depend upon contact bearing, when assembled during fabrication, shall have at least 75% of the entire contact area in full bearing and the separation at the edges of the joint shall not exceed 0.60 mm unless otherwise stipulated by the Engineer.

13.12.4 The vector sum of factored longitudinal and transverse shear loads shall not exceed the factored resistances given in Table 5(b), unless an ultimate strength analysis is used that is acceptable to the Engineer.

13.12.5 Plug and slot welds shall be considered only to provide shear resistance in the plane of the connected parts.

Table 5(a)
Base Metal and Matching[1] Electrode Ultimate Strengths

Matching Electrode[2] Ultimate Strengths (MPa)	G40.21M Grades						
	260	300	350	380	400	480	700
410	X	X[4]					
480	X	X	X[3]	X			
550					X		
620						X	
820							X

Notes:

(1) For matching condition of ASTM steels see Table 11-1 or Table 12-1 of CSA Standard W59.

(2) Ultimate strengths have been determined from electrode classification numbers.

(3) For unpainted applications using "A" or "AT" steels where the deposited weld metal shall have similar atmospheric corrosion resistance and/or similar colour characteristics to the base metal, the requirements of Clause 5.2.1.4 and 5.2.1.5 of CSA Standard W59 shall apply.

(4) For HSS only.

Table 5(b)
Factored Resistances of Welds

Type of Weld	Type of Load and Orientation	Requirements for matching*	Factored Resistance
Complete Joint Penetration Groove Welds	Tension or compression parallel to axis of weld*	Matching condition, or if electrode classification is lower or higher than matching. See note (2).	
	Tension normal to effective throat	Matching is mandatory.	Same as for base metal
	Compression normal to effective throat	Matching is mandatory.	
	Shear on effective throat	Matching condition	
		Electrode classification lower than matching. See note (1).	The smaller of (b) base metal $V_r = 0.66\phi F_y A_m$ or (b) weld metal $V_r = 0.67\phi_w X_u A_w$
Partial Joint Compression Groove Welds	Tension or compression parallel to axis of weld*	Matching condition, or if electrode classification is lower or higher than matching. See note (2).	Same as for base metal
	Compression normal to axis of weld	Matching condition is mandatory.	See Note (4)

Table 5(b) continued

Type of Weld	Type of Load and Orientation	Requirements for matching*	Factored Resistance
Partial Joint Penetration Groove Welds	Tension normal to axis of weld	Matching condition, or if electrode classification is lower than matching use X_u of lower electrode.	The smaller of (a) base metal (lesser of) (i) $T_r = \phi F_y A_m$ (ii) $T_r = 0.85\phi F_u A_m$ or (b) weld metal $V_r = 0.67\phi_w X_u A_w$
	Shear	Matching condition, or if electrode classification is lower than matching use X_u of lower electrode	The smaller of (a) base metal $V_r = 0.67\phi F_y A_m$ (b) Weld metal $V_r = 0.67\phi X_u A_w$
Fillet Welds	Tension or compression parallel to axis of weld	Matching condition or if classification is lower than matching. See note (2).	Same as calculated for base metal.
	Tension or compression normal to axis of weld	For matching condition use X_u of matching electrode classification.	The smaller of (a) base metal $V_r = 0.67\phi F_y A_m$ or (b) weld metal $V_r = 0.67\phi_w X_u A_w$
	Shear		
Plug and Slot Welds	Shear	For electrode classification lower than matching use X_u of lower electrode. For electrode classification one designation higher than matching use X_u of matching electrode.	

* For CSA G40.21M steels and their matching electrode classification refer to Table 5(a).

Notes:
1. Electrodes of strength lower than that shown for the corresponding base metal grades of Table 5(a) may be used for complete joint penetration groove welds between webs and flanges of girders transferring shear loads.

Table 5(b) continued

2. If shear is transferred between components, the design value is the smaller of that for the base metal or the weld metal.

3. For HSS, the provisions of Appendix L of CSA Standard W59, Welded Steel Construction (Metal Arc Welding) may be used.

4. Factored resistances of joints are calculated as for base metal but on an area, $A = A_m$ + area of base metal in contact bearing.

5. The following symbols are used in this Table:

A_w = the effective area of weld in shear
A_w shall be taken as follows:
(a) For plug and slot welds: A_w = area of faying surface
(b) For all others: A_w = size of effective throat of weld x length of weld

A_m = the area of fusion face
A_m shall be taken as follows:

(a) For fillet welds: A_m = effective size x length of weld

(b) For complete joint penetration groove welds:
(i) Butt Joint
A_m = thickness of base metal x length of weld
(ii) T-Joint
A_m = size of fusion face in base metal x length of weld

(c) For partial joint penetration groove welds:
(i) Joints in Tension
A_m = applicable area of base metal normal to tensile load
(ii) All others, the lesser of
1. A_m = size of fusion face in base metal x length of weld
2. A_m = thickness of base metal x length of weld.

(d) For Plug and Slot Welds: A_m = area of faying surface

F_u = specified ultimate tensile strength of base metal

F_y = specified minimum yield stress, yield point or yield strength of base metal

F_v = allowable stress

X_u = ultimate strength as rated by electrode classification number

$\phi_w = 0.67$

14. Cold Formed Steel Structural Members

14.1 Scope

14.1.1 Clause 14 shall apply to the design of structural members, cold formed to shape from carbon or low-alloy, sheet or strip steels, used for load-carrying purposes in buildings.

14.1.2 Where the provisions of Clause 14 differ from the provisions of other clauses of this Standard the provisons of Clause 14 shall govern for cold formed steel structural members.

14.2 Definitions and Symbols

14.2.1 **Definitions**
The following definitions apply to Clause 14.

Cold forming means the shaping of flat rolled steel at ambient temperature to form a structural section;

Effective width ratio means the ratio of the effective width (b) to the thickness (t) of the element. Effective width ratio shall be determined in accordance with requirements of Clause 14.4.4.2;

Flange of a section in bending means the flat width including any intermediate stiffeners plus the adjoining corners;

Flat width ratio means the ratio of the flat width (w) to the thickness (t) of the element;

Multiple stiffened element means an element that is adequately stiffened at both edges according to Clause 14.4.5.2 and also stiffened by means of intermediate stiffeners which are parallel to the direction of stress and which conform to the requirements of Clause 14.4.5.3;

Partially-effective element means an element for which the effective width is less than the flat width;

Point symmetric section means a section symmetric about its centroid;

Stiffened element means a flat element of which both edges parallel to the direction of stress are supported by stiffening means conforming to the requirements of Clause 14.4.4.2;

Sub-element of a multiple-stiffened element means the portion of such an element between adjacent stiffeners or between a web and intermediate stiffener or between an edge and intermediate stiffener;

Unstiffened element means a flat element with one longitudinal free edge;

Virgin steel means steel in the condition prior to cold forming (e.g., coiled or cut to length);

Web of a section in bending means that portion which joints two flanges. It is taken as the flat length measured in the plane of the web excluding the corners.

14.2.2 **Symbols**

The following symbols apply to Clause 14. Other symbols found in Clause 14 have previously been defined in Clause 5.

A_e Effective cross-sectional area

a Distance between web centerlines; distance between attachments

B Force in bracing; stud spacing

B_L Limiting value of B

B_r Factored resistance to web crippling of the webs of flexure members

b Effective design width; distance between flange centerlines; width of largest leg of an angle

C_1 Factor used in calculating shear strain in sheathing of wall studs

c Allowable amount of curling; distance from the centroid of a member to its extreme compressive fibre

D Ratio of mean diameter to thickness of hollow circular section

D_A Number of 90° corners in the flange of a section in bending or in the entire cross-section of a compression or tension member. If angles other than 90° are used, D_A is the sum of the bend angles divided by 90°

d Clear distance between flanges

	about the shear center
S_{xc}	Compressive section modulus of entire section about the major axis
s	Spacing
T_s	Strength of connection in tension
t	Thickness of thinnest connected sheet
t_s	Equivalent thickness
t_w	Effective throat thickness of a fillet weld based on minimum leg size
W*	Ratio of the centerline length of the flange of a member in bending or of the entire section of a tension or compression member to the thickness
w	Width of an element exclusive of fillets; flat width
w_f	Projection of flange from inside face of web of a channel or half the distance between webs for box or U-type sections
w'	Width of flange projection beyond the web for I-beam and similar sections or half the distance between webs for box or U-type sections; for flanges of I-beams and similar sections stiffened by lips at the outer edges, w' shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.
x_o	Distance from shear center to centroid of section
γ	Shear strain in sheathing
$\bar{\gamma}$	Limit shear strain in sheathing under factored load
ε	Yield strain
μ	Poisson's ratio (0.33 assumed)
θ	Angle between web and plane of bearing surface
σ	Limit stress related to shear strain in wall studs

Φ' Post buckling factor

ϕ_c Resistance factor for connections

ϕ_o Resistance factor for web crippling in beams having other than a single unreinforced web

ϕ_u Resistance factor for web crippling in beams with a single unreinforced web

14.3 Material Standards

14.3.1 General

Acceptable material and product standards and specifications for use under this clause are listed in Clauses 14.3.2 and 14.3.3.

14.3.2 Structural Steel

CSA G40.21-M,
Structural Quality Steels;

ASTM A36,
Structural Steel;

ASTM A242,
High Strength Low-Alloy Structural Steel;

ASTM A283,
Low and Intermediate Tensile Strength Carbon Steel Plates of Structural Quality;

ASTM A572,
High-Strength Low-Alloy Columbium-Vanadium Steels Structural Quality;

ASTM A588,
High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick;

14.3.3 Sheet Steel

ASTM A446M,
Steel Sheet, Zinc Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality (Grades A, B, C, D, F);

ASTM A570,
Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality;

ASTM A606,
Sheet Steel and Strip, Hot-Rolled and Cold-Rolled,

High-Strength, Low-Alloy with Improved Corrosion Resistance;

ASTM A607,
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy Columbium and/or Vanadium;

ASTM A611,
Steel, Cold-Rolled Sheet, Carbon, Structural (Grades A, B, C & D);

ASTM A715,
Sheet Steel and Strip, Hot-Rolled, High-Strength Low-Alloy, with Improved Formability;

ASTM A816M,
Specification for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, High Strength Low-Alloy;

CSSBI 101-M,
Zinc Coated Structural Quality Steel Sheet for Roof and Floor Deck.

14.3.4 Physical Properties

The physical properties used for design purposes in Clause 14 shall be taken as:

Young's modulus E	203 000 MPa
Shear modulus G	78 000 MPa
Poisson's ratio (μ)	0.33
Mass density	7 850 kg/m^3
Coefficient of linear thermal expansion	$11.7 \times 10^{-6}/°C$

14.4 General Design Considerations

14.4.1 Cold Work of Forming

14.4.1.1 General

Utilization of cold work of forming is optional and if used shall be in accordance with Clause 14.4.1.2, and only applied to the following additional Clauses of this Standard:

(a) Clause 14.5.3, Flexural Members- Single Web;

(b) Clause 14.5.5, Concentrically Loaded Compression Members;

(c) Clause 14.5.6, Members Subject to Combined Axial Load and Bending;

(d) Clause 14.5.8, Wall Studs.

14.4.1.2 Axially Loaded Tension Members; Tension Flanges of Beams; Fully Effective Axial Compression Members; Fully Effective Compression Flanges of Beams

The yield strength (F'_y) of (a) axially loaded tension members, (b) the tension flanges of flexural members, (c) fully effective axially loaded compression members and (d) the compression flanges of flexural members whose stiffened elements are not subject to a reduction in effective area as required by Clause 14.4.4.2, shall be determined by one of the following methods:

(a) Full section tensile tests as specified in Clause 14.8.3.1;

(b) $F'_y = F_y + 5D_A\ (F_u - F_y)/W^*$

14.4.1.3 Axial Compression Members and Compression Flanges which are not Fully Effective

The yield strength (F'_y) of axially loaded compression members and the compression flanges of flexural members not conforming to Clause 14.4.1.2 shall be taken as the tensile yield strength of the virgin steel specified by the relevant specification of Clause 5.

14.4.2 Maximum Allowable Flat Width Ratios for Compression Elements

Maximum allowable overall flat width ratios w/t, disregarding intermediate stiffeners and taking as t, the actual thickness of the element, unless otherwise specified shall be as follows:

(a) Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by:

(i) Simple lip bent at right angles to the element..............................60;

(ii) A more effective kind of stiffener.......90;

(b) Stiffened compression element with both longitudinal edges connected to a web or flange element (U-type or box-type sections)...........................500;

(c) Unstiffened compression element..............60.

Note: 1. Unstiffened compression elements (see Clause 14.4.2(c)) that have flat width ratios exceeding approximately 30 and stiffened compression elements (see Clause 14.4.2(b)) that have flat width ratios exceeding approximately 250 are likely to develop noticeable deformations under specified loads without detriment to the load-carrying ability.

2. Additional specific limits may be given in certain clauses.

14.4.3 Maximum Allowable Web Depths

The web slenderness ratio, h/t, of the webs of flexural members shall not exceed the following limitations:

(a) For unreinforced webs: $h/t \leq 200$;

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Clause 14.5.4

(i) When using bearing stiffeners only:	$h/t \leq 260$;
(ii) When using bearing stiffeners and intermediate stiffeners:	$h/t \leq 300$;

Where a web consists of two or more sheets, the ratio, h/t, shall be computed for the individual sheets.

14.4.4 Properties of Sections

14.4.4.1 General

Properties of sections such as cross sectional area, moment of inertia, section modulus and radius of gyration, shall be determined in accordance with conventional methods of structural design.

14.4.4.2 Effective Width

When w/t exceeds $290\sqrt{\frac{k}{f}}$, w/t shall be replaced by an effective width ratio, b/t, as given in Clause 14.6.2.1. For stiffened compression elements, that portion of the total width which is considered removed to arrive at the effective width shall be located symmetrically about the centroid of the element. For unstiffened compression elements, the portion of the total width which is considered removed to arrive at the effective width shall be located at the unstiffened edge of the element.

14.4.4.2.1 Compression Elements and Sub-elements of Multiple-Stiffened Elements

The effective width ratio of compression elements, for strength determinations at the factored load level and for deflection or vibration determination at the specified load level, shall be taken as:

$$\frac{b}{t} = 428 \sqrt{\frac{k}{f}} \left[1 - \frac{94}{(w/t)} \sqrt{\frac{k}{f}}\right] - R$$

where

k = 4.0 for stiffened compression elements,
k = 0.5 for unstiffened compression elements,
f = stress in compression element computed on the basis of the effective width.

R is given by:

(i) R = 0.1 (w/t) - 6 when (w/t >60)
(ii) R = 0 when (w/t < 60)

When the element or sub-element is stiffened at each edge by means of a web or flange, R may be taken as zero for all values of w/t.

14.4.4.2.2 Effective Area of Stiffeners

For computing the effective structural properties of a member having compression sub-elements or elements subject to the correction factor R of Clause 14.4.4.2.1 in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area A_e as follows:

For 60 < w/t < 90

$$A_e = \left(3 - 2(b/w) + (b/t)/30 - (w/t)/30\right)A$$

For w/t > 90

$$A_e = (b/w)A$$

Where A_e and A refer to the stiffener and w and b refer to the member.

14.4.4.2.3 Unusually Short Spans Supporting Concentrated Loads

Where the span of a flexural member is less than 30w', and the member carries one concentrated load, or several loads spaced farther apart than 2 w', the effective width of any flange, whether in tension or compression, shall be limited by the ratio given in Table 6.

Table 6
Short Wide Flanges
Maximum Allowable Ratio of Effective Width

L/w'	30	25	20	18	16	14	12	10	8	6
Ratio b/w	1.00	0.96	0.91	0.89	0.86	0.82	0.78	0.73	0.67	0.55

where L = full span for simple spans; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.

w' = width of flange projection beyond the web for I-beam and similar sections or half the distance between webs for box or U-type sections. For flanges of I-beams and similar sections stiffened by lips at the outer edges; w' shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

14.4.5 Stiffeners for Compression Elements

14.4.5.1 General

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener. See Clause 14.4.4.2.2 for effective area of stiffener.

14.4.5.2 Edge Stiffeners

In order that a flat compression element may be considered a stiffened compression element, it shall be stiffened along one longitudinal edge parallel to the direction of stress by a web, and along the other edge by a web, lip or other stiffener with a moment of inertia equal to the greater of

(i) $I = 9t^4$
(ii) $I = \left(2(w/t) - 13\right)t^4$

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required overall depth of the lip shall be taken as

$d_\ell = t\left(24(w/t) - 156\right)^{1/3}$ but not less than 4.8t.

A simple lip shall not be used as an edge stiffener for any element having a flat width ratio greater than 60.

14.4.5.3 Intermediate Stiffeners for Multiple-Stiffened Elements

In order that a flat compression element may be considered to be a multiple-stiffened element, it shall be stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners parallel to the direction of stress. Each such intermediate stiffener shall have a moment of inertia equal to twice that specified for edge stiffeners in Clause 14.4.5.2. Furthermore,

(a) If the spacing of stiffeners between two webs is such that the flat width ratio (w/t) of any of the sub-elements between stiffeners is larger than b/t only two intermediate stiffeners (those nearest each web) shall be considered effective;

(b) If the spacing of stiffeners between a web and an edge stiffener is such that the flat width ratio, w/t, of any of the sub-elements between stiffeners is larger than b/t, only the intermediate stiffener nearest the web shall be considered effective; and

(c) If intermedfiate stiffeners are spaced so closely that the flat width ratio, w/t between stiffeners does not exceed b/t, all the stiffeners may be considered effective. For this case, the flat width ratio and effective width ratio of the entire multiple-stiffened element, shall be determined using a width equal to the total width between webs or from web to edge stiffener, and a thickness equal to

$$t_e = t\left[\frac{w}{2}p + \sqrt{\frac{3I_s}{pt^3}}\right]^{1/3}$$

14.4.6 Curling of Flanges

Where a flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange towards the neutral axis, the gross width w/t of the flange shall be as follows:

(a) For tension flanges, unstiffened compression flanges and fully effective stiffened compression flanges:

$$w/t < \frac{450}{\sqrt{F_y}}\left(\frac{dc}{t^2}\right)^{1/4}$$

(b) For stiffened compression flanges that are not fully effective:

$$w/t < \frac{225}{t}\sqrt{\frac{dc}{F_y}}$$

where w is the gross width of flange projecting beyond the web or half the distance between webs for box or U-type beams.

Note: The allowable amount of curling will vary with different kinds of sections and must be established by the designer. (An amount of curling in the order of 5 per cent of the depth of the section is usually not considered excessive.)

14.5 Member Resistance

14.5.1 General

To meet the strength requirements of this clause all factored resistances shall be greater than or equal to all factored resistances of cold formed steel members as determined in Clause 7.2 and ϕ shall be taken as

(a) Tension, compression and shear...... ϕ = 0.90

(b) Web crippling in beams:
 (i) Single unreinforced web........ ϕ_u = 0.75
 (ii) Other webs..................... ϕ_o = 0.67

(c) Connections....................... ϕ_c = 0.67

14.5.2 Axial Tension

14.5.2.1 The factored tensile resistance, T_r, developed by a member subjected to an axial tensile force shall be as given in Clause 13.2(a).

14.5.2.2 For angles connected by bolts in one leg the net section shall be reduced by 0.70 of the area of the outstanding leg. For channels connected by bolts in the web the net section shall be reduced by the area of the outstanding flanges.

14.5.3 Flexural

14.5.3.1 General

The factored moment resistance, M_r, of straight flexural members shall be taken as:

$$M_r = \phi S F_c$$

Where continuous lateral support is provided, $F_c = F_y$. Where continuous lateral support is not provided, F_c shall be computed in accordance with

Clause 14.5.3.2 or 14.5.3.3.

14.5.3.2 Single-Web Members (I, Z, or Channel Shaped Members)

14.5.3.2.1 When bending is about the centroidal axis perpendicular to the web for either I-shaped sections or symmetrical channel-shaped sections, F_c shall be taken as:

(a) When $0.833(F_{be} + F_t) > 0.5F'$

$$F_c = F' - \left[\frac{0.25(F')^2}{0.833(F_{be} + F_t)}\right] \text{ but not greater than } F_y$$

(b) When $0.833(F_{be} + F_t) \leq 0.5F'$

$$F_c = 0.833(F_{be} + F_t)$$

where

$F_{be} = \pi^2 EdI_{yc}/\omega L^2 S_{xc}$

$F_t = 0.333GAt^2/\omega dS_{xc}$

$F' = 1.11F_y$

ω see Clause 13.7.4

14.5.3.2.2 For point-symmetrical Z-shaped sections bent about the centroidal axis perpendicular to the web:

(a) When $0.833(F_{be} + F_t) > 0.5F'$

$$F_c = F' - \left[\frac{0.50(F')^2}{0.833\ (F_{be} + F_t)}\right] \text{ but not greater than } F_y$$

(b) When $0.833(F_{be} + F_t) \leq 0.5F'$

$$F_c = 0.833\ (0.5F_{be} + 0.5F_t)$$

14.5.3.2.3 For channels and Z-shaped members with unstiffened flanges, F_c shall be taken as:

$$F_c = \frac{0.5\pi^2 E}{12(1-\mu)^2 (w/t)^2} \text{ but not greater than } F_y$$

14.5.3.3 Closed Box Flexural Members

When bending is about the major axis of the section, F_c shall be taken as:

$$F_c = \frac{\pi}{LS_{xc}} \sqrt{EI_y GJ}$$

where

J = St. Venant torsion constant, $= \dfrac{2(ab)^2}{(a/t_1) + (b/t_2)}$

where

a = distance between web centrelines
b = distance between flange centrelines
t_1 = thickness of flanges
t_2 = thickness of webs

14.5.3.4 Bending in Webs

The maximum bending stress shall not exceed F_y in tension or compression and in addition the maximum bending stress in compression shall not exceed:

$$F_c = \Phi F_{bw}$$

where

$$F_{bw} = \frac{183\ 000k}{(h/t)^2}$$

$$\Phi = \alpha_1 \alpha_2 \alpha_3 \alpha_4 > 1.0$$

$$\alpha_1 = 0.017(h/t) - 0.79$$

$$\alpha_2 = 0.462 \left|\frac{f_c}{f_t}\right| + 0.538$$

For beams with stiffened compression flanges

$$\alpha_3 = 1.16 - 0.16 \frac{(w/t)}{290\sqrt{k/F_y}} < 1 \text{ when } \frac{(w/t)}{290\sqrt{k/F_y}} < 2.25$$

$$\alpha_3 = 0.8 \text{ when } \frac{(w/t)}{290\sqrt{k/F_y}} > 2.25$$

For beams with unstiffened compression flanges

$$\alpha_3 = 0.84 - 0.019 \frac{(w/t)}{290\sqrt{k/F_y}}$$

$$\alpha_4 = \left(\frac{F_y}{406}\right) + 0.10$$

$$k = 4 + 2\left(1 + \left|\frac{f_t}{f_c}\right|\right)^3 + 2\left(1 + \left|\frac{f_t}{f_c}\right|\right)$$

f_t = maximum tensile bending stress in web

f_c = maximum compressive bending stress in web

14.5.3.5 Shear in Webs

The factored shear resistance V_r developed by the web of a flexural member subjected to shear shall be taken as:

$$V_r = \phi A_w F_s$$

where F_v is the lesser of:

(a) $F_s = F_y/\sqrt{3}$

(b) (i) For $h/t \leq 618 \sqrt{\frac{k_v}{F_y}}$ .... $F_s = \frac{296\sqrt{k_v F_y}}{(h/t)}$

(ii) For $h/t > 618 \sqrt{\frac{k_v}{F_y}}$ ...... $F_s = \frac{183\ 000\ k_v}{(h/t)^2}$

where

A_w = ht as defined in Clause 13.4.1

Where the web consists of two or more sheets, each sheet shall be considered to carry its share of the shear.

14.5.3.6 **Combined Bending and Shear Resistance in Webs**

For webs subject to both bending and shear stresses, the member shall be proportioned such that the following limits are observed:

$$\frac{M_f}{M_r} \leq 1$$

$$\frac{V_f}{V_r} \leq 1$$

$$\left(\frac{M_f}{M_r}\right)^2 + \left(\frac{V_f}{V_r}\right)^2 \leq 1.0$$

For beam webs with transverse stiffeners satisfying the requirements of Clause 14.5.4 the member may be proportioned such that the following limits are observed:

$$\frac{M_f}{M_r} \leq 1$$

$$\frac{V_f}{V_r} \leq 1$$

$$0.6\left(\frac{M_f}{M_r}\right) + \left(\frac{V_f}{V_r}\right) < 1.3 \quad \text{when} \quad \frac{M_f}{M_r} > 0.5$$

$$\text{and} \quad \frac{V_f}{V_r} > 0.7$$

14.5.3.7 Web Crippling

Bearing stiffeners shall be provided when h/t > 200 and when the factored concentrated load or reaction exceeds the factored compressive resistance of the webs as given in Tables 7, 8, and 9 where B_r is the resistance offered by one solid web. The bearing resistance of two or more sheets is the sum of the individual resistances.

For built-up I-members, or similar sections, the distance between the connector and member flange shall be kept as small as practicable.

Reactions or loads are classified as single flange loading when the clear distance measured longitudinally between the bearing edges of adjacent loads on opposite flanges exceeds 1.5h and as opposite flange loading when this distance is equal to or less than 1.5h.

Reactions or loads are classified as end reactions when the distance from the edge of the bearing to the end of the member is equal to or less than 1.5h and as interior reactions when this distance exceeds 1.5h.

The bearing resistance of two channels connected back to back and for similar sections which provide a high degree of restraint against rotation of the web, such as I-sections made by welding two angles to a channel shall be taken as given in Table 7.

14.5.3.8 Combined Web Crippling and Bending

Unreinforced flat webs of shapes subjected to a combination of bending and web crippling shall be designed to meet the following requirements:

$$P_f/B_r + M_f/M_r \leq 1.3$$

This interaction equation is not applicable to multi-web deck sections.

Table 7

I-Beams

Flange Loading	Reaction or Load	Bearing Resistance, B_r
Single	End	$B_r = \phi_o C_2(10 + 1.25\sqrt{N/t})t^2 F_y$
	Interior	$B_r = \phi_o C_1(0.88 + 0.063t)(15 + 3.25\sqrt{N/t})t^2 F_y$
Opposite	End	$B_r = \phi_o C_4(0.64 + 0.16t)(10 + 1.25\sqrt{N/t})t^2 F_y$
	Interior	$B_r = \phi_o C_3(0.82 + 0.079t)(15 + 3.25\sqrt{N/t})t^2 F_y$

The above formulas apply when $\frac{r}{t} < 4$, $\frac{N}{t} < 200$ and $N/h < 1$

Table 8
Shapes Having Single Webs

Flange Loading	Reaction or Load	Bearing Resistance, B_r
		Stiffened Flanges
Single	End	$B_r = 10\phi_u(1.33 - 0.33k)(1.15 - 0.15(r/t))$ $(1 + 0.01(N/t))(1 - 0.0018(h/t))t^2F_y$
		Unstiffened Flanges
		$B_r = 6.6\phi_u(1.33 - 0.33k)(1.15 - 0.15(r/t))$ $(1 + 0.01(N/t))^* (1 - 0.0013(h/t))t^2F_y$
	Interior	$B_r = 16\phi_u(1.22 - 0.22k)(1.06 - 0.06(r/t))$ $(1 + 0.007(N/t))^{**}(1 - 0.0014(h/t))t^2F_y$
Opposite	End	$B_r = 7.4\phi_u(1.33 - 0.33k)(1.15 - 0.15(r/t))$ $(1 + 0.01(N/t))(1 - 0.0023(h/t))t^2F_y$
	Interior	$B_r = 16\phi_u(1.22 - 0.22k)(1.06 - 0.06(r/t))$ $(1 + 0.01(N/t))(1 - 0.0029(h/t))t^2F_y$

* When $N/t > 60$, the factor $(1 + 0.01N/t)$ may be increased to $(0.71 + 0.015N/t)$.

** When $N/t > 60$, the factor $(1 + 0.007N/t)$ may be increased to $(0.75 + 0.011N/t)$.

The above formulas apply when $r/t < 4$, $N/t < 200$ and $N/h < 1$.

Table 9
Deck Sections

Flange Loading	Reaction or Load	Bearing Resistance, B_r
Single	End	$B_r = 14\ \phi_u(\sin\theta)\ (1 - 0.1k)\ (1 - 0.1\sqrt{r/t})$ $(1 + 0.005(N/t))\ (1 - 0.002h/t)t^2F_y$
	Interior	$B_r = 17\ \phi_u(\sin\theta)\ (1 - 0.1k)$ $(1 - 0.075\sqrt{r/t})\ (1 + 0.005(N/t))$ $(1 - 0.001h/t)t^2F_y$
Opposite	End	$B_r = 11\ \phi_u(\sin\theta)\ (1 - 0.1k)\ (1 - 0.1\sqrt{r/t})$ $(1 + 0.01(N/t))\ (1 - 0.002(h/t))t^2F_y$
	Interior	$B_r = 18\ \phi_u(\sin\theta)\ (1 - 0.2k)$ $(1 - 0.03\sqrt{r/t})\ (1 + 0.01(N/t))$ $= (1 - 0.0015(h/t))t^2F_y$

The above formulas apply to decks when $r/t < 10$ and $N/t < 200$.

Note: For hat sections both legs must be fastened to prevent spreading.

In Tables 7, 8 and 9

$C_1 = (1.49 - 0.53k) > 0.6$

$C_2 = 1 + \frac{(h/t)}{750} < 1.2$

$C_3 = \frac{1}{k}$ when $h/t < 66.5$;

$C_3 = \left(1.1 - \frac{(h/t)}{665}\right)/k$ when $h/t > 66.5$

$C_4 = 0.98 - \frac{(h/t)}{865}/k$

$k = F_y/230$

θ = angle between plane of web and plane of bearing surface > 45° but no more than 90°

14.5.4 **Transverse Stiffeners for Beam Webs**

14.5.4.1 **Bearing Stiffeners**

Bearing stiffeners shall bear against the flanges or flange through which they receive their loads. Stiffeners shall be designed as columns in accordance with Clause 16.5.1 assuming the column section to comprise the pair of stiffeners and a centrically located strip of the web less than or equal to eighteen times its thickness at interior stiffeners, or strip not equal to more than 10 times its thickness when the stiffeners are located at the end of the web. The effective column length KL shall be taken as not less than 3/4 the length of the stiffeners. Bearing stiffeners shall be connected to the web in accordance with clause 14.6 so as to develop the full force required to be carried by the stiffener into the web or vice versa.

The flat width ratio, w/t, of the stiffened and unstiffened elements of transverse stiffeners shall not exceed $780/\sqrt{F_y}$ and $228/\sqrt{F_y}$, respectively.

14.5.4.2 **Intermediate Stiffeners**

14.5.4.2.1 Intermediate transverse stiffeners when used shall be spaced to suit the shear resistance determined from the formula given in Clause 14.5.3.5 and the maximum distance between stiffeners shall be as given in Clause 16.6.2.

Intermediate transverse stiffeners may be furnished singly or in pairs. The moment of inertia of the stiffener or pair of stiffeners if so furnished must be at least equal to the greater of

(i) $I = (h/50)^4$

(ii) $I = 5ht^3[h/a - 0.7(a/h)]$

taken about an axis in the plane of the web. The gross area of intermediate stiffeners, or pairs of stiffeners if so furnished, shall be taken as

$$A_s > \frac{at}{2}\left[1 - \frac{a/h}{(a/h) + \left(1 + (a/h)^2\right)^{1/2}}\right] CYD$$

where

a = distance centre to centre of adjacent stiffeners (i.e. panel length)

$$C = 1 - \frac{310\ 000k_v}{F_y(h/t)^2} \text{ but not less than } 0.10$$

Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel
D = stiffener factor
= 1.0 for stiffeners furnished in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners
k_v = shear buckling coefficient (see Clause 13.4.1)
F_y = specified minimum yield point of web steel

The factored resistance of stamped or rolled-in transverse stiffeners shall be determined by tests in accordance with Clause 14.8.

14.5.5 Axially Loaded Compression Members

14.5.5.1 General

The requirements of Clause 14.5.5 apply only to material of 4.5 mm or less in thickness. Members formed from thicker material shall be designed in accordance with the requirements of Clause 13.3. Compressive resistances of singly-symmetric shapes are given in Clause 14.5.5.3; hollow structural sections are given in Clause 14.5.5.4; built-up members are given in Clause 14.5.5.6 and axially loaded wall studs are given in Clause 14.5.8. Except as given in Clause 14.5.5.5 in which $w/t \leq 150$ for stiffened compression elements and $w/t \leq 35$ for unstiffened compression elements C_r shall be taken as:

$$C_r = \phi A_e F_o$$

(a) For $F_p > F_y/2$ $\qquad F_o = F_y - \dfrac{F_y^2}{4F_p}$

(b) For $F_p < F_y/2$ $\qquad F_o = F_p$

where F_p is defined in Clauses 14.5.5.2 to 14.5.5.3.

14.5.5.2 Sections not Subject to Torsional-Flexural Buckling

For I-shapes, closed cross section shapes, and any other shapes which can be shown not to be subject to torsional-flexural buckling, F_p shall be taken as:

$$F_p = 0.833\pi^2 E/(KL/r)^2$$

14.5.5.3 Singly-Symmetric Shapes

14.5.5.3.1 For singly-symmetric open sections, such as plain and lipped channels and single or double plain and lipped angles which may be subject to torsional-flexural buckling, F_p shall be taken as the lesser of:

(a) F_p as defined in Clause 14.5.5.2

(b) $F_p = \frac{1}{2\beta} \{F_s + F_t - \sqrt{(F_s + F_t)^2 - 4\beta F_s F_t}\}$

where:

$F_s = 0.833\pi^2 E/(KL/r_s)^2$

$F_t = \frac{0.833}{Ar_0^2} [GJ + \frac{\pi^2 EC_w}{(KL)^2}]$

$\beta = 1 - (x_0/r_0)^2$

$r_0^2 = r_x^2 + r_y^2 + x_0^2$

KL = effective length with respect to torsion

x_0 = distance from shear centre to centroid along x-axis

$J = \frac{1}{3} \sum \ell t^3$

ℓ = middle line length of member segments

KL/r = maximum slenderness ratio with respect to the x or y axis

r_s = radius of gyration about the axis of symmetry

14.5.5.3.2 For channel, Z shape and single angle sections with unstiffened flanges shall be designed in accordance with Clauses 14.5.5.1 and 14.5.5.3, except that the factored resistance shall be limited additionally as follows:

$C_r = \phi A \frac{76\ 400}{(w/t)^2}$

This additional limit shall be waived if the channel is fully restrained with respect to torsion and flexural buckling about the asymmetric axis.

14.5.5.3.3 **Point-Symmetric Shapes Which May be Subject to Torsional Buckling**

Point-symmetric open shapes such as cruciform sections which are not braced against twisting shall meet the requirements of Clause 14.5.5.1 taking F_p as the lesser of:

(a) F_p as defined in 14.5.5.2

(b) $F_p = F_t$ where F_t is defined in 14.5.5.3.1.

14.5.5.4 Hollow Structural Sections

The design of hollow structural section compression members that comply with the requirements of CSA Standard G40.20, General Requirements for Rolled or Welded Structural Quality Steel, shall meet the requirements of Clause 13.3.2.

14.5.5.5 Other Sections

For non-symmetric shapes whose cross-sections do not have any symmetry, either about an axis or a point, and for sections formed with any stiffened element whose flat width ratio exceeds 150 or any unstiffened elements whose flat width ratio exceeds 35, the factored compressive resistance shall be determined by rational analysis. Alternatively, compression members composed of such shapes may be tested in accordance with Clause 14.8.

14.5.5.6 Built-up Members

14.5.5.6.1 For compression members composed of two or more elements connected together at discrete points, such as double angles and battened channels, subjected to buckling about the composite axis, F_p shall be taken as

$$F_p = 0.833 \left[\frac{\pi^2 E}{(KL/r)^2 + (a/r_z)^2}\right]$$

where

KL/r = overall slenderness ratio of the complete section about the composite axis.

a/r_z = slenderness ratio of the individual elements between points of connection, about an axis parallel to the composite axis.

14.5.5.6.2 Each discrete connection must be capable of transmitting a longitudinal shear force between the bars of 5 per cent of the force in one element.

14.5.5.6.3 For torsional-flexural buckling of singly symmetric sections F_p shall be taken as given in Clause 14.5.5.3.1 with F_s replaced by F_p as given in Clause 14.5.5.6.1.

14.5.6 Combined Axial Load and Bending

14.5.6.1 Axial Compression and Bending

Members required to resist bending moments and an axial compressive force shall be proportioned so that:

(a) $\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$

where $C_r = \phi A_e F_y$ and,

$$(b)\ \frac{C_f}{C_r} + \frac{\omega_x M_{fx}}{M_{rx}\left(1 - \frac{C_f}{C'_{ex}}\right)} + \frac{\omega_y M_{fy}}{M_{ry}\left(1 - \frac{C_f}{C'_{ey}}\right)} \leq 1.0$$

where

C_r is defined in Clause 15.5.5

M_r is defined in Clause 15.5.3

$$C'_e = \frac{2\ 003\ 500 A_e}{(KL/r)^2}$$

ω_x, ω_y = as defined in Clause 13.7.2

When $C_f/C_r \leq 0.15$ the value of $\left(1 - \frac{C_{ex}}{C_y}\right)$, $\left(1 - \frac{C_{ey}}{C_y}\right)$,

ω_x and ω_y may be assumed to be equal to one.

14.5.6.2 **Axial Tension and Bending**

Members required to resist both bending moments and an axial tensile force shall be proportioned in accordance with 13.8. The moment resistance, M_r, shall be based on the net section.

14.5.7 **Single Angles Loaded Through One Leg**

For single angles loaded at each end through the same leg by bolts or welds, the factored compressive resistance shall be as given in Clause 14.5.5.1 where

$$F_p = 0.833 \left\{ \frac{\pi^2 E}{\left[\left(\frac{KL}{r}\right)^2 + \left(\frac{5b}{t}\right)^2\right]} \right\}$$

where r = minimum radius of gyration
b = leg width
K = 0.8 for single bolt connections and
= 0.7 for two bolt connections or welds

14.5.8 **Wall Studs**

The factored compressive resistance of a stud may be computed on the basis that wall materials or sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided the stud, wall material, and attachments comply with the following requirements.

(a) Both ends of the stud shall be braced against rotations about the longitudinal stud axis and translations perpendicular to the stud axis;

however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis.

(b) The cladding shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

14.5.8.1 Studs in Compression

14.5.8.1.1 For studs having identical sheathing material with design shear rigidity, $\bar{q}B$, attached to both flanges, and neglecting any rotational restraint provided by the sheathing material the compression resistance shall be taken as:

$$C_r = \phi A_e F_o$$

where F_o is the least of:

(a) F_o as given in Clause 14.5.6.1 with KL equal to two times the distance between fasteners

(b) $F_o = F_y - \dfrac{F_y^2}{4F_p}$ when $F_p > F_y/2$

$F_o = F_p$ when $F_p \leq F_y/2$

(c) $F_o = 0.833\sigma$, where σ is determined to satisfy the requirement that the shear strain, γ, in the sheathing corresponding to the stress, σ, shall not exceed the allowable shear strain of the sheathing, $\bar{\gamma}$ given in Table 10.

$$\bar{\gamma} \geq \frac{\pi}{L}\left[C_1 + E_1 \frac{d}{2}\right]$$

where F_p is given in Clause 14.5.8.1.2 and C_1 and E_1 are given in Clause 14.5.8.1.3. To initiate the iterative calculations required to establish the strain compatibility of γ and $\bar{\gamma}$, σ should initially be taken as F_o as computed in Clause 14.5.8.1.1(b).

14.5.8.1.2 (a) For singly symmetric channel and C sections F_p shall be taken as the lesser of:

(i) $F_p = 0.833\left[F_{ey} + \left(\dfrac{\bar{q}B}{A}\right)\right]$

(ii) $F_p = \dfrac{0.833}{2\beta}\left[\left(F_{ex}+F_{tQ}\right) - \sqrt{\left(F_{ex}+F_{tQ}\right)^2 - 4\beta F_{ex}F_{tQ}}\right]$

(b) For Z-sections F_p shall be taken as the lesser of:

(i) $F_p = 0.833\left[F_t + \dfrac{\bar{q}Bd^2}{4Ar_o^2}\right]$

$$\text{(ii) } F_p = \frac{0.833}{2}\left[\left(F_{ex} + F_{ey} + \left(\frac{\bar{q}B}{A}\right)\right) - \sqrt{\left(F_{ex} + F_{ey} - \left(\frac{\bar{q}B}{A}\right)\right) - 4\left(F_{ex}F_{ey} + F_{ex}\left(\frac{\bar{q}B}{A}\right) - F_{exy}^{2}\right)}\right.$$

(c) For doubly symmetric I-sections F_p shall be taken as the lesser of:

$$\text{(i) } F_p = 0.833\left(F_{ey} + \frac{\bar{q}B}{A}\right)$$

$$\text{(ii) } F_p = 0.833F_{ex}$$

In the above formulas

$$F_{ey} = \frac{\pi^2 E}{(L/r_y)^2}$$

$$F_{ex} = \frac{\pi^2 E}{(L/r_x)^2}$$

$$F_{exy} = \frac{\pi^2 EI_{xy}}{AL^2}$$

$$F_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{L^2}\right]$$

$$F_{tQ} = F_t + \left(\frac{\bar{q}Bd^2}{4Ar_o^2}\right)$$

where

$\bar{q}$ = shear rigidity per mm of stud spacing based on sheathing on both sides for two wallboards per mm of stud given in Table 10
B = stud spacing
A = cross sectional area of stud
β = $1 - (x_o/r_o)^2$
x_o = distance from shear center to centroid along principal x-axis
r_o = polar radius of gyration of cross-section about shear center

14.5.8.1.3 The values for C_1 and E_1 shall be taken as follows:

(a) For Single Symmetric Channel and C Sections

$$C_1 = \frac{\sigma C_o}{F_{ey} - \sigma + \bar{Q}_a}$$

$$E_1 = \frac{\sigma\left[(F_{ex} - \sigma)(r_o^2 E_o - x_o D_o) - \sigma x_o(D_o - x_o E_o)\right]}{(F_{ex} - \sigma)\ r^2\ (F_{tQ} - \sigma) - (\sigma x_o)^2}$$

(b) For Z Sections

$$C_1 = \frac{\sigma[C_o(F_{ex} - \sigma) - D_o F_{exy}]}{(F_{ey} - \sigma + \overline{Q}_a)(F_{ex} - \sigma) - (F_{exy})^2}$$

$$E_1 = \frac{\sigma E_o}{F_{tO} - \sigma}$$

(c) For I Sections

$$C_1 = \frac{\sigma C_o}{F_{ey} - \sigma + \overline{Q}_a}$$

$$E_1 = 0$$

Table 10
Sheathing Parameters(1)

Wall Board(2)	$\overline{q}_o$ (3) N/mm	$\overline{\gamma}$ mm/mm
9.5 to 15.9 mm thick gypsum	525	0.008
Lignocellulosic board	263	0.009
Fiberboard (regular or impregnated)	158	0.007
Fiberboard (heavy impregnated)	315	0.0010

Notes:
(1) The values given were established from small-scale tests and are subject to the following important limitations: All values are for wall boards on both sides of the wall assembly. All fasteners are No. 6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent, at 150 to 300 mm spacing.
(2) All wall boards are 12.7 mm thick, except as noted.
(3) $\overline{q} = \overline{q}_o(2 - s/300)$
where s = the fastener spacing

For other types of sheathings, $\overline{q}_o$ and $\overline{\gamma}$ may be determined conservatively from representative small-specimen tests as described by published documented methods. Wall board parameter values $\overline{q}_o$ and γ, determined from representative full-scale tests described by published documented methods, may also be used instead of the small-scale test values given in Table 10.

where

C_o, E_o, and D_o are initial column imperfections which shall be assumed to be at least

C_o = L/350 in a direction parallel to wall

D_o = L/700 in a direction parallel to wall

E_o = L/(d x 10 000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled location

In case $\sigma > 0.5F_y$, then in the definitions for F_{ey}, F_{ex}, F_{exy}, and F_{tQ}, the parameters E and G are to be replaced in Clause 14.5.8.1.1(ii) by E' and G', respectively, given as:

$$E' = 4E_o(F_y - \sigma)/(F_y)^2$$

$$G' = G(E'/E)$$

14.5.8.1.4 Studs with sheathing on one side only; or with unidentical sheathing; or when the rotational restraint is not neglected; or any combination of the above shall be designed in accordance with the same basic principles of analysis used in deriving the provisions in Clause 14.5.8.1.1.

14.5.8.2 **Studs in Axial Compression and Bending**

The design strength of studs subject to combined axial compression and bending shall be taken as:

$$\frac{C_f}{C_r} + \frac{M_{fx}}{\left[1 - \frac{C_f}{C'_{rx}}\right] M_{rx}} \leq 1.0$$

when $\frac{C_f}{C_r} \leq 0.15$, the following formula may be used in lieu of the above:

$$\frac{C_f}{C_r} + \frac{M_f}{M_{rx}} \leq 1.0$$

where

C_r = compression resistance under concentric loading according to Clause 14.5.5

M_{rx} = bending resistance where bending only exists (Clauses 14.5.2 and 14.5.3),

$C'_{rx} = \phi A_e[\pi^2 E/(L/r_x)^2]$

14.6 Connections

14.6.1 Fastening Devices
Any suitable mechanical fastener, special device, or other means may be used to join component parts provided that the type of fastening device is compatible with the service conditions.

14.6.2 Welded Connections

14.6.2.1 General
An allowance shall be made for the effect of welding on the mechanical properties of the member. This effect shall be determined by tests on full section specimens which contain the weldment within the gauge length.

14.6.2.2 Arc Welds

14.6.2.2.1 General Requirements
Arc welds shall meet the following requirements depending on the minimum thickness, t, of the connected parts.

(a) $t > 3.5$ mm
Fusion welds on steel shall conform to the requirements of CSA Standard W59, "Welded Steel Construction (Metal-Arc Welding)".

(b) $0.70 \text{ mm} \leq t \leq 3.5$ mm
Fusion welds on steel shall conform to the requirements contained herein, and in all cases the welding shall be performed in accordance with CSA Standard W59.

(c) $t < 0.70$ mm
Fusion welds made on steel shall be considered to have no structural value.

14.6.2.2.2 Butt Welds
The resistance of butt welds in tension or compression shall be taken as the lower strength of base metal joined. The weld shall have full penetration.

14.6.2.2.3 Arc-Spot Welds (Puddle Welds)
Arc spot welds shall be used only for the welding in the flat position of sheet steel to a supporting member. Type E410XX or E480XX electrodes shall be

used to melt through the sheet steel to fuse with the plate. The weld shall be round in shape with a visible nominal diameter of 20 mm. The thickness of the structural supporting member shall be at least 2.5 times the steel sheet thickness. The minimum edge distance measuring from the centreline of the weld to the end or boundary of the connected member shall not be less than 25 mm. The steel sheet shall be of weldable type and shall have a tensile yield strength of 230 MPa or greater. The resistance per weld shall be taken as:

(a) For shear

$$V_r = 10^3\phi_c(20t - 5)$$

(b) For tension

$$T_r = 10^3\phi_c(5.6t - 1)$$

Note: The factored resistances apply only to sheet thicknesses from 0.70 mm to 1.52 mm.

14.6.2.2.4 Fillet Welds

Fillet welds may be made in any position. The factored shear resistance of fillet welds shall be taken as:

(a) For welds parallel to the direction of loading

When $L/t < 25$ $V_r = \phi_c\left(1 - 0.01 \frac{L}{t}\right) tLF_u$

When $L/t > 25$ $V_r = \phi_c 0.75tLF_u$

(b) For welds perpendicular to the direction of loading

$$V_r = \phi_c tLF_u$$

14.6.2.2.5 Flare Groove Welds

Flare groove welds may be made in any welding position. Sheet to sheet connections may be made with flare-V and flare-bevel groove welds and sheet to thicker steel member connections may be made with flare-bevel groove welds. The factored shear resistance of welds shall be governed by the thickness, t, of the sheet steel adjacent to the welds. The factored shear resistance shall be taken as:

(a) For loads applied perpendicular to the axis of the weld

(i) Flare-bevel groove welds

$$V_r = 0.8\phi_c tLF_u$$

(ii) Flare-V groove welds

Loads applied perpendicular to the axis of the weld have not been considered.

(b) For loads applied parallel to the axis of the weld

when $t \leq t_w < 2t$ or $d_\ell < L$

$$V_r = 0.75\phi_c tLF_u$$

when $t_w \geq 2t$ or $d_\ell > L$

$$V_r = 1.5\phi_c tLF_u$$

where t_w is the lesser of the two throats shown in Figure 1.

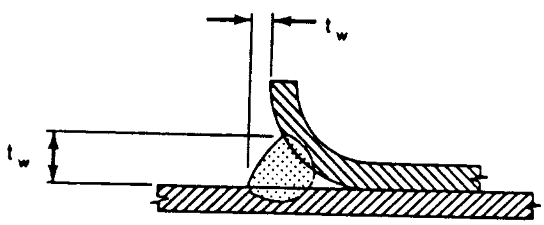


Figure 1.

14.6.2.3 **Resistance Welds**
The factored shear resistance per spot weld for sheets joined by spot welding shall be taken as:

$$V_r = 4000\phi_c t^{1.5}$$

This equation applies to welds in sheets between 0.40 mm and 6 mm thick.

14.6.3 **Connections Made by Bolts, Rivets and Screws**

14.6.3.1 **Shear Resistance**

14.6.3.1.1 The shear resistance for bolts, V_r, shall be taken as given in Clauses 13.10 and 13.11.

14.6.3.1.2 For screws and special fasteners, for which Clause 14.6.3.1.1 cannot be applied, the factored resistance shall be taken as ϕ_c times the manufacturer's certified ultimate shear resistance in the condition specified.

14.6.3.2 Bearing Resistance of Bolted Connections.

14.6.3.2.1 For bolted connections with washers under both bolt head and nut the bearing load on the area, dt, shall not exceed the bearing resistance as follows:

(a) For an inside sheet of double shear connection

(i) For $F_u/F_y > 1.15$ $B_r = 3.75\phi_c F_u A$

(ii) For $F_u/F_y < 1.15$ $B_r = 3.37\phi_c F_u A$

(b) For single shear, and outside sheets of double shear connection

$B_r = 3.375\phi_c F_y A$

14.6.3.2.2 For bolted connections without washers under both bolt head and nut, or with only one washer where $0.91 < t < 4.76$ the bearing load on the area, dt, shall not exceed the bearing resistance as follows:

(a) For an inside sheet of double shear connection

For $F_u/F_y > 1.15$ $B_r = 3.37\phi_c F_u A$

(b) For single shear, and outside sheets of double shear connection

For $F_u/F_y > 1.15$ $B_r = 2.5\phi F_u A$

14.6.3.2.3 When $t > 4.76$ mm Clause 13.10 shall be used. For conditions not defined in Clauses 14.6.3.2.1 and 14.6.3.2.2 stresses shall be determined on the basis of test data using a resistance factor of 0.67.

14.6.3.3 Tension Resistance of Net Section

The net cross section may be considered to be fully effective in tension for symmetrically applied forces.

14.6.3.3.1 Clause 14.6.3.3 shall only apply when $t < 4.76$ mm. When $t > 4.76$ mm Clause 13.10 shall apply.

14.6.3.3.2 The tension resistance, T_r, on the net cross section of a bolted connection with washers under both head and nut shall not exceed $\phi F_y A_n$ nor shall it exceed the following:

(a) For double shear connection T_r shall be taken as the lesser of

(i) $T_r = (1.0 - 0.9r + 3rd/s)1.2\phi_c F_u A$

(ii) $T_r = 1.25\phi_c F_u A$

(b) For single shear connection T_r shall be taken as the lesser of

(i) $T_r = (1.0 - 0.9r + 3rd/s)1.12\phi_c F_u A$

(ii) $T_r = 1.12\phi_c F_u A$

14.6.3.3.3 The tension resistance, T_r, on the net cross section of a bolted connection without washers under bolt head and nut, or with only one washer shall not exceed $\phi F_y A_n$ nor shall it exceed the following:

(i) $T_r = (1.0 - 4 + 2.5rd/s)1.12\phi_c F_u A$

(ii) $T_r = 1.12\phi_c F_u A$

where r = the force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0.2, it may be taken equal to zero.

14.6.3.4 **Minimum Edge Distance and Spacing**
The distance from the centre of a fastener to the edge shall not be less than 1.5d. The distance between fasteners, centre-to-centre, shall not be less than 2.5d.

14.6.4 **Connections in Built-up Members**

14.6.4.1 The number of fasteners joining elements together to form a beam or column shall be sufficient to transfer the shear forces developed. The spacing of fasteners shall be such that buckling of the elements between fasteners is prevented.

14.6.4.2 The compressive resistance of built-up members shall be determined in accordance with Clause 14.5.5.6.

14.6.4.3 Fasteners in beams, at the point of application of concentrated loads shall be capable of transferring any such load applied to one element only.

The connection at points of application of concentrated loads of double channel beams shall be capable of resisting a force, T_r, tending to separate the flanges and shall be taken as:

$$T_r = \frac{P_f m}{2g}$$

For double channel beams subject to a uniformly distributed load, P_f shall be taken as:

$P_f = 3sq_f$

Maximum fastener spacing for beams shall not exceed L/4.

14.6.5 Spacing of Connections in Compression Elements

The spacing, s, in line of stress, of welds, rivets, or bolts connecting a compression cover plate or sheet to a non-integral stiffener or other element shall not exceed

(a) the spacing required to transmit the shear between the connected parts based on connection strength

(b) $s = 680t/\sqrt{f}$

where f is the limit stress in the cover plate or sheet

(c) $s = 3w$, or $500t/\sqrt{F_y}$, whichever is greater

where

w = flat width of the narrowest unstiffened compression element in the portion of the cover plate or sheet which is tributary to the connections.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds plus 13 mm. In all other cases the spacing shall be taken as the centre-to-centre distance between connections.

Note: The requirements of Clause 14.6.5 do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

14.7 Bracing

14.7.1 General

14.7.1.1 General requirements for the bracing of compression members and compression flanges of beams and compression chords of trusses are given in Clause 20.

14.7.1.2 The provisions of Clause 14.7.2 apply to compression members and flexural members of symmetric section in which the applied loading does not induce twist.

14.7.1.3 The provisions of Clause 14.7.3 apply to flexural members, such as channel and Z sections, in which load applied in the plane of the web induces twist.

14.7.2 Sections Which are Symmetric Relative to the Plane of Loading

14.7.2.1 Discrete Braces

The factored resistance of a brace shall be at least 2.0 percent of the factored compressive force in the member at the braced location.

14.7.2.2 Bracing by Deck, Slab or Sheathing

The requirements of Clause 20.3.2 shall be met.

14.7.3 Channels and Z-Sections Used in Flexure

14.7.3.1 Bracing shall be provided to resist twisting of channel and Z sections loaded in flexure in the plane of the web.

14.7.3.2 Bracing when Both Flanges are Braced by Deck or Sheathing Material

The factored resistance of the attachment shall meet the requirements of Clause 20.3.2.

14.7.3.3 Bracing When One Flange is Braced by Deck or Sheathing Material

14.7.3.3.1 The factored resistance of the attachment shall meet the requirements of Clause 20.3.2.

14.7.3.3.2 Discrete braces shall be provided to restrain the flange which is not braced by the deck slab or sheathing.

14.7.3.3.3 The spacing of discrete braces shall be in accordance with Clauses 14.7.3.4.1 and 14.7.3.4.2.

14.7.3.4 Bracing When Neither Flange is Braced by Deck or Sheathing Material

The following provisions for the spacing and design of discrete braces shall apply.

14.7.3.4.1 Spacing of Braces

Braces shall be attached both to the top and bottom flanges of the sections at the ends and at intervals not greater than one-quarter of the span length in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces, unless it can be shown by rational analysis or testing, taking into account both the effects of lateral and torsional displacements, that fewer braces can be used. If one-third or more of the total load on the beams is concentrated over a length of one-twelfth or less of the span, an additional brace shall be placed at or near the center of this loaded length.

14.7.3.4.2 Design of Flange Braces

Each intermediate brace, at the top and bottom flange, shall be designed to resist a lateral force, a function of the braced length, a, as follows:

(a) For a uniformly loaded beam

$B = 1.5K'aq$

(b) For concentrated loads when

$0 < x < 0.3a \qquad P = K'P$

$0.3 < x < 1.0a \qquad P = 1.43(1 - \frac{x}{a})K'P$

where

x = distance from concentrated load P to brace
K' = m/d for channels
$K' = I_{xy}/I_x$ for Z-sections

Braces shall be designed to avoid local crippling at the points of attachment to the member.

End braces shall be designed for one-half the above forces.

14.7.3.5 Moment Resistance

The moment resistance of channels and Z-beams braced at intermediate points according to the requirements of Clauses 14.7.2.2 and 14.7.2.3, shall be determined in accordance with Clause 14.5.5, using, a, as the unbraced length.

14.7.4 Laterally Unbraced Box Beams

For closed box-type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed $17000/F_y$.

14.8 Testing

14.8.1 General

Testing facilities shall be suitable for the type of test. Tests may be made at a manufacturer's or an independent testing facility. Test results and reports for type B and C tests shall be certified by a professional engineer. The provisions of this section do not apply to steel deck diaphragms and to composite steel comoponents or composite steel assemblies.

14.8.2 **Types of Tests**

Tests are classified as follows.

Type A - Cold Formed Steel Properties

Full section tests to determine the modified mechanical properties of steel after cold working or cold forming for the utilization of the change in strength as permitted in Clause 14.4.1.2.

Type B - Performance Tests

Structural performance tests to establish the limit states of structural elements or assemblies for which the composition or configuration are such that calculation of the load resistance or deformation cannot be made in accordance with the provisions of this Standard.

Type C - Confirmatory Tests

Confirmatory tests to verify the limit states of structural elements or assemblies designed in accordance with the provisions of this Standard. In no case shall the resistance established by these tests be taken to be greater than that computed in accordance with the provisions of this Standard.

14.8.3 **Test Procedures**

14.8.3.1 **Type A**

14.8.3.1.1 Tensile testing procedures shall be performed in accordance with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370.

14.8.3.1.2 Comprehensive yield strength determinations shall be made by means of compression tests of short specimens of the section. The compressive yield strength shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross-section area or the strength defined by one of the following methods:

(i) for sharp yielding steel the yield strength shall be determined by the autographic diagram method or by the total strain under load method. When the total strain under load method is used, there shall be evidence that the yield strength so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method.

(ii) for gradual yielding steel the yield strength shall be determined by the strain under load method or by the 0.2 percent offset method. When the total strain under load method is used, there shall be evidence that the yield strength so determined agrees within 5 percent with the yield strength which would be determined by the 0.2 percent offset method.

14.8.3.1.3 Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield strength shall be determined for the flanges only. In determining such yield strengths, tests shall be made on specimens cut from the section. Each such specimen shall consist of one complete flange plus a portion of the web such that the specimen is fully effective.

14.8.3.1.4 For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tonnes nor less than 30 tonnes of each section. For this purpose, lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat or blow.

14.8.3.1.5 At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield strength of the section when subjected to the type of stress under which the member is to be used.

14.8.3.2 **Type B**

14.8.3.2.1 Testing procedures shall be subject to the approval of the Regulatory Authority. Due consideration shall be given to the duration of load and boundary conditions in service of the elements and assemblies.

14.8.3.2.2 The Resistance, R, shall be taken as $0.85\phi Q$ where Q is the test value for the limit state as established in this clause and ϕ is the appropriate Resistance Factor as given in Clause 14.5.1. If the critical property (yield strength, ultimate tensile strength, modulus of elasticity, etc.) of the steel from which the test sections are formed is larger than the specified value, the test results shall be adjusted to the specified minimum value of this property for the steel which the manufacturer intends to use. When the critical property is less than the minimum specified no such adjustment shall be made.

Variations between the tested and specified values of the geometric properties shall also be taken into account.

14.8.3.2.3 The test value for the limit state, Q, shall be established based on the mean values resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all tests does not exceed ± 10 percent. If such deviation from the mean exceeds 10 percent, at least three more tests of the same kind shall be made. The average of the three lowest values of all tests made shall then be regarded as the test value for the limit state, Q.

14.8.3.3 Type C

Confirmatory tests shall be performed and analyzed as for type B tests and shall meet the following criteria:

(a) the factored resistance, ϕR shall be greater than or equal to the effect of factored loads

(b) the specified deformation limit shall be greater than or equal to that due to specified loads.

14.9 Fabrication

14.9.1 Forming, Cutting, Punching and Drilling

Members shall be formed at ambient temperature by a method which does not result in work hardening to an extent that would limit the intended service and, where applicable, which does not result in damage to protective coatings which have been applied to the unformed material. Components may be cut by slitting, shearing, sawing, or flame cutting. Holes for fasteners may be punched or drilled.

14.9.2 Fastenings

Steel components may be assembled by means of welding or by the use of mechanical fasteners such as bolts, rivets or screws. Where dissimilar metals are fastened together attention shall be paid to electrical separation and the selection of suitable fasteners, recognizing the possibility of galvanic corrosion.

Fastenings such as metal stitching, clinching and structural adhesives also may be used where suitable. The strength of fastenings shall be established by test in accordance with Clause 14.8.3.2, unless values are specified elsewhere in this Standard.

14.9.3 Profiles and Distortion

Cold formed steel members shall be made to the full dimensions claimed by the manufacturer. Care shall be taken not to stretch, bend or otherwise distort parts of cold formed members except as a necessary feature of the cold forming operation.

14.10 Protection

14.10.1 Protection During Assembly, Storage, and Erection

Cold formed members shall be adequately protected from corrosion and deformation during assembly, storage and erection.

14.10.2 Uncoated Stock

All uncoated steel stock shall be stored in a dry place before processing and, except for weathering grades, shall be protected by a rust inhibitive coating immediately after processing.

14.10.3 Coatings

Cold formed members other than those made of weathering grades of steel, shall be protected against corrosion by means of paint, zinc, aluminum, porcelain enamel, or other effective means, either singly or in combination.

14.10.4 Preparation of Surfaces for Coatings

Before applying a protective coating, steel surface shall be dry, clean, and free from dirt, grease, loose or heavy scale and rust. When preparing welded assemblies for painting, the area at or near welds shall be thoroughly cleaned. After surfaces have been cleaned, a protective coating shall be applied as soon as practicable and before noticeable oxidation of clean surfaces occurs.

14.10.5 Adequate Adhesion of Protective Coatings

Careful consideration shall be given to the selection of a protective coating system to ensure that all procedures are compatible and will be such as to ensure adequate adhesion of the coating film.

15. Fatigue

15.1 General

15.1.1 In addition to meeting the requirements of Clause 15 for fatigue, any member or connection shall also meet the requirements for the static load conditions using the factored loads.

15.1.2 Members and connections subject to fatigue loading shall be designed, detailed and fabricated so as to minimize stress concentrations and abrupt changes in cross-section.

15.1.3 Specified loads for the design of members or connections shall be used for all fatigue calculations.

15.1.4 A specified load less than the maximum specified load but acting with a greater number of cycles may govern and shall be considered.

15.1.5 Plate girders with $h/w > 3150/\sqrt{F_y}$ shall not be used under fatigue conditions.

15.1.6 Slotted holes shall not be used in bolted connections in members subject to fatigue.

15.2 Life

For guidance in determining the number of cycles the life of the building should be assumed to be not less than 50 years unless otherwise stated.

15.3 10 000 Cycles of Load

When a load is expected to be applied not more than 10 000 times during the life of the structure, no special considerations beyond those in Clause 15.1.2 need apply.

15.4 Over 10 000 Cycles of Load

When a load is expected to be applied more than 10 000 times in the life of the structure, the loaded members, connections, bolts, and welds shall be proportioned so that the probability of fatigue failure is acceptably small. In such cases the design should be based on the best available information as to the fatigue characteristics of the materials and components to be used. In the absence of more specific information, Clause 15.5 provides guidance in proportioning members and parts. Fatigue resistance shall be provided only for those loads considered to be repetitive and hence contributing to fatigue. Often the magnitude of a repeated load is less than the maximum static load which the member or part would be designed to sustain.

15.5 Allowable Range of Stress in Fatigue

When this Clause is used as the basis for design, the members, connections, bolts and welds shall be proportioned so that the computed range of stress does not exceed the allowable range of stress F_{sr} given in Table 11(a) for the appropriate type and location of material shown in Table 11(b). The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress. Members subject to a range of stress involving only compression need not be designed for fatigue. The information in Tables 11(a) and 11(b) is shown diagramatically in Appendix J.

Table 11(a)
Allowable Ranges of Stress in Fatigue

Category (see Table 11(b) and Figure J2)	F_{sr} (MPa) For 100 000 Cycles	For 500 000 Cycles	For 2 000 000 Cycles	Over 2 000 000 Cycles
A	415	250	165	165
B	310	190	125	110
C	220	130	90	70*
D	185	110	70	48
E	145	85	55	32
F	110	65	40	18
W	115	85	65	48

* Except for transverse stiffener welds on girder webs or flanges, where 83 MPa may be used.

15.6 Secondary Effects

Secondary stresses, stresses due to deformations, and stresses due to out-of-plane movements are potential sources of fatigue cracks. Caution is therefore advised in detailing structures which are subjected to repetitive loads and in which these sources of stresses may be present.

Table 11(b)
Description of Design Conditions for Various Joint Classifications

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Plain Material	S1	Base metal with rolled or cleaned surfaces. Flame cut edges with a surface roughness not exceeding 1000 (25 μm) as defined by CSA Standard B95.	A	1, 2
Built-up Members	S2	Base metal and weld metal in members without attachments, built-up of plates or shapes connected by continuous complete or partial penetration groove welds or by continuous fillet welds, parallel to the direction of applied stress.	B	3,4,5
	S3	Base metal and weld metal along the length of horizontal stiffeners and cover plates connected by continuous complete or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	B	7
	S4	Base metal at toe of transverse stiffener welds on girder webs or flanges subjected to calculated flexural stress.	C	6
	S5	Base metal at end of longitudinal stiffeners.	E	7
	S6	Base metal at end of partial length welded cover plates narrower than the flange, having square or tapered ends, with or without welds across the ends.		
		Flange thickness ≤ 20 mm	E	7
		Flange thickness > 20 mm	F	

Built-up Members continued ...

Table 11(b) continued

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Built-up Members	S7	Base metal at end of partial length cover plates wider than the flange having square ends with welds across the ends.		
		Flange thickness < 20 mm	E	7
		Flange thickness > 20 mm	F	
Complete Joint Penetration Grooves	S8	Base metal and weld metal at complete penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by non-destructive examination.*	B	8, 10
	S9	Base metal and weld metal in or adjacent to complete penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 4 in 10, with grinding in the direction of applied stress, and weld soundness established by non-destructive examination.* A 600 mm curved radius transition shall be used for CAN3-G40.21M-700Q and 700QT steel.	B	11, 12
	S10	Base metal and weld metal in or adjacent to complete penetration groove welded splices, involving equal widths and/or thicknesses of material, or involving transitions having slopes no greater than 4 in 10 when, in either case, reinforcement is not removed and when weld soundness is established by non-destructive examination.*	C	8,10,11,12

Complete Joint Penetration Grooves continued ...

Table 11(b) continued

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Complete Joint Penetration Grooves	S11	For base metal at details attached to flanges or webs by groove welds subjected to transverse or longitudinal loading - regardless of detail length except for conditions as covered by Note (1) in tabulation for Example 13 the stress range categories shall be as shown in Fig. J2 in the tabulation for the sample example. Besides being dependent on transition radius, the stress range categories, in the case of flange connections subject to transverse loading, are also a function of relative thickness of material and whether or not groove weld reinforcement is removed.	See Tabulation in Ex. 13 Fig. J2	13
Fillet Welded Connections	S12	Base metal at intermittent fillet welds.	E	
	S13	Base metal adjacent to fillet welded attachments where length L of the attachment in direction of stress is less than 50 mm.	C	6,14,15,16
	S14	Base metal at details attached by fillet welds subjected to longitudinal loading only when the detail length, L in direction of stress is between 50 mm and 12 times the plate thickness, but less than 100 mm and the transition radius R is less than 50 mm.	D	15

Fillet Welded Connections continued ...

Table 11(b) continued

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Fillet Welded Connections	S15	For base metals at details attached to webs by fillet welds subjected to transverse and/or longitudinal loading regardless of detail length the stress range categories shall be as shown in Figure J2 in the tabulation for the sample Example. Shear stress on the throat of fillet welds shall be governed by stress range category "W".	See Tabulation in Ex. 13 Fig. J2	13
	S16	Except for cover plates (S6, S7) and details attached to webs (S15) base metal at end of details 100 mm or longer attached by fillet welds where the length of weld is in the direction of stress.	E	17
Fillet Welds	S17	Shear stress on throat of fillet welds.	W	17
Stud Type Shear Connections	S18	Shear stress on the nominal area of stud shear connectors.	W	
Mechanically Fastened Connections	S19	Base metal at gross section of high-strength bolted slip-resistant connections, except axially loaded joints which induce out-of-plane bending in connected material.	B	9
	S20	Base metal at net section of high-strength bolted bearing-type connections and other mechanically fastened joints.	B	9
	S21	Base metal at net section of bolted connections other than high-strength bearing-type.	D	9

15.7 **Single Load Path Structures**

Structures in which the failure of a single element could result in collapse or catastrophic failure require special attention when fatigue cracking is a possibility. When Clauses 15.1 to 15.6 are followed in the design, the structure shall be subject to periodic inspection and maintenance. Alternatively, the permissible stress ranges shall be limited to 0.80 times those given in Table 11(a).

16. Beams and Girders

16.1 **Proportioning**

Beams and girders consisting of rolled shapes (with or without cover plates), hollow structural sections, or fabricated sections shall be proportioned on the basis of the properties of the gross section or the modified gross section as noted below. No deduction shall be made for fastener holes in webs or flanges unless the reduction of flange area by such holes exceeds 15 percent of the gross flange area, in which case the excess shall be deducted. The effect of openings other than holes for fasteners shall be considered in accordance with Clause 16.10.

16.2 **Rotational Restraint at Points of Support**

Beams and girders shall be restrained against rotation about their longitudinal axes at points of support.

16.3 **Reduced Moment Resistance of Girders With Thin Webs**

When the web slenderness ratio, h/w, exceeds $1810/\sqrt{M_f/\phi S}$ the flange must meet the width-thickness ratios of Class 3 sections of Clause 11 and the factored moment resistance of the beam or girder, M'_r, shall be determined by:

$$M'_r = M_r \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{w} - 1810/\sqrt{M_f/\phi S}\right)\right]$$

where

M_r = factored moment resistance determined by Clause 13.5 or 13.6 but not to exceed ϕM_y.

16.4 **Flanges**

16.4.1 Flanges of welded girders preferably shall consist of a single plate or a series of plates joined end-to-end by complete penetration groove welds.

16.4.2 Flanges of bolted girders shall be proportioned so that the total cross-sectional area of cover plates does not exceed 70 per cent of the total flange area.

16.4.3 Fasteners or welds connecting flanges to webs shall be proportioned to resist horizontal shear forces due to bending combined with any loads which are transmitted from the flange to the web other than by direct bearing. Spacing of fasteners or intermittent welds in general shall be in proportion to the intensity of the shear force and shall not exceed the maximum for compression or tension members as applicable, in accordance with Clause 18.

16.4.4 Partial length flange cover plates shall be extended beyond the theoretical cut-off point and the extended portion shall be connected with sufficient fasteners or welds to develop a force in the cover plate at the theoretical cut-off point not less than:

$$P = \frac{AM_{fc}y}{I_g}$$

where

P = required force to be developed in cover plate
A = area of cover plate
M_{fc} = moment due to factored loads at point of theoretical cut-off
y = distance from centroid of cover plate to neutral axis of cover-plated section
I_g = moment of inertia of cover-plated section

Additionally, for welded cover plates, the welds connecting the cover plate termination to the beam or girder shall be designed to develop the force P defined above within a length a' measured from the actual end of the cover plate, determined as follows:

(a) a' = the width of cover plate when there is a continuous weld equal to or larger than three-fourths of the cover plate thickness across the end of the plate and along both edges in the length a'

(b) a' = 1.5 times the width of cover plate when there is a continuous weld smaller than three-fourths of the cover plate thickness across the end of the plate and along both edges in the length a'

(c) a' = 2 times the width of cover plate when there is no weld across the end of the plate but continuous welds along both edges in the length a'

16.5 Bearing Stiffeners

16.5.1 Pairs of bearing stiffeners on the webs of single-web beams and girders shall be required at points of concentrated loads and reactions wherever the bearing resistance on the web is exceeded (see Clause 16.8). Bearing stiffeners shall be required also at unframed ends of single-web girders having web slenderness ratios greater than $1100/\sqrt{F_y}$. Box girders may employ diaphragms designed to act as bearing stiffeners.

16.5.2 Bearing stiffeners shall bear against the flange or flanges through which they receive their loads, and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns in accordance with Clause 13.3, assuming the column section to comprise the pair of stiffeners and a centrally located strip of the web equal to not more than 25 times its thickness at interior stiffeners, or a strip equal to not more than 12 times its thickness when the stiffeners are located at the end of the web. The effective column length, KL, shall be taken as not less than three-fourths of the length of the stiffeners in computing the ratio KL/r. Only that portion of the stiffeners outside of the angle fillet or the flange-to-web welds shall be considered effective in bearing. Angle bearing stiffeners shall not be crimped. Bearing stiffeners shall be connected to the web so as to develop the full force required to be carried by the stiffener into the web or vice versa.

16.6 Intermediate Transverse Stiffeners

16.6.1 Intermediate transverse stiffeners when used shall be spaced to suit the shear resistance determined from the formula given in Clause 13.4; except that at girder end panels or at panels containing large openings, the smaller panel dimension, a or h, shall not exceed $1150w/\sqrt{V_f/\phi A_w}$ where V_f is the largest shear in the panel.

16.6.2 The maximum distance between stiffeners, when stiffeners are required, shall not exceed the values shown in Table 12. Closer spacing may be required in accordance with Clause 16.6.1.

16.6.3 Intermediate transverse stiffeners may be furnished singly or in pairs. Width-thickness ratios shall conform to Clause 11. The moment of inertia of the stiffener, or pair of stiffeners if so furnished, shall be not less than $(h/50)^4$ mm^4 taken about an axis in the plane of the web. The gross area of intermediate stiffeners, or pairs of stiffeners if so furnished, shall be given by the expression

$$A_s > \frac{aw}{2}\left[1 - \frac{a/h}{\sqrt{1 + (a/h)^2}}\right] CYD$$

Table 12
Maximum Intermediate Transverse Stiffener Spacing

Web Slenderness Ratio (h/w)	Maximum Distance Between Stiffeners (a) in Terms of Clear Web Depth (h)
Up to 150	3h
Over 150	$\frac{67\ 500h}{(h/w)^2}$

where

a = distance centre-to-centre of adjacent stiffeners (i.e. panel length)
w = web thickness
h = web depth
$C = 1 - \frac{310\ 000k_v}{F_y(h/w)^2}$ but not less than 0.10
Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel
D = stiffener factor
= 1.0 for stiffeners furnished in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners
k_v = shear buckling coefficient (see Clause 13.4.1)
F_y = specified minimum yield point of web steel.

When the greatest shear, V_f, in an adjacent panel is less than that permitted by Clause 13.4.1, this gross area requirement may be reduced in like proportion by multiplying by the ratio V_f/V_r.

16.6.4 Intermediate transverse stiffeners shall be connected to the web for a shear transfer per pair of stiffeners (or per single stiffener when so furnished), in newtons per millimetre of web depth (h), not less than $1 \times 10^{-4} hF_y^{3/2}$; except that when the largest computed shear V_f in the adjacent panels is less than V_r computed by Clause 13.4.1 this shear transfer may be reduced in the same proportion. However the total shear transfer shall in no case be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener. Fasteners connecting intermediate transverse stiffeners to the web shall be spaced not more than 300 mm on centre. If intermittent fillet welds are used, the clear distance between welds shall not exceed 16 times the web thickness or 4 times the weld length.

16.6.5 When intermediate stiffeners are used on only one side of the web, the stiffeners shall be attached to the compression flange. Intermediate stiffeners used in pairs shall have at least a snug fit against the compression flange. When stiffeners are cut short of the tension flange the distance cut short shall be equal to or greater than 4 times but not greater than 6 times the girder web thickness. Stiffeners preferably shall be clipped to clear girder flange-to-web welds.

16.7 Lateral Forces

The flanges of beams and girders supporting cranes or other moving loads shall be proportioned to resist any lateral forces produced by such loads.

16.8 Web Crippling

Bearing stiffeners shall be provided where the factored concentrated load or reactions exceed the factored compressive resistances of webs of rolled beams and welded plate girders at the web toe of the flange-to-web fillets computed as follows:

(a) For interior loads

$$B_r = 1.25\phi w\ (N + 2k)F_y$$

(b) For end reactions

$$B_r = 1.25\phi w\ (N + k)F_y$$

where

w = web thickness
N = length of bearing (N shall be not less than k for end reactions)
k = distance from outer face of flange to web toe of flange-to-web fillet

16.9 Stability of Thin Webs

The sum of all loads on the compression edge of the web plate resulting from concentrated and distributed loads bearing directly, or through a flange plate, and not supported by bearing stiffeners, shall not exceed the factored resistances as calculated below:

(a) When the flange is restrained against rotation

$$B_r = \phi \frac{115\ 000}{(h/w)^2} \left[5.5 + \frac{4}{(a/h)^2}\right] A$$

(b) When the flange is not restrained against rotation

$$B_r = \phi \frac{115\ 000}{(h/w)^2} \left[2 + \frac{4}{(a/h)^2}\right] A$$

where for distributed loads A is equal to the panel length times the web thickness and for concentrated loads and loads distributed over partial length of a panel, A equals the web thickness times the lesser panel dimension, a or h.

16.10 **Openings**

16.10.1 Except as provided in Clause 16.1, the effect of all openings in beams and girders shall be considered in the design. At all points where the factored shear or moments at the net section would exceed the capacity of the member at that point adequate reinforcement shall be added to the member to provide the required strength and stability.

16.10.2 Unreinforced circular openings may be located in the web of unstiffened prismatic compact beams or girders without considering net section properties provided that:

(a) The specified design load for the member is uniformly distributed;

(b) The section has an axis of symmetry in the plane of bending;

(c) The openings are located within the middle third of the depth and the middle half of the span of the member;

(d) The spacing between the centres of any two adjacent openings, measured parallel to the longitudinal axis of the member, is a minimum of 2.5 times the diameter of the larger opening;

(e) The factored maximum shear at the support does not exceed 50 per cent of the shear resistance of the section.

16.10.3 If the forces at openings are determined by an elastic analysis, the procedure adopted shall be in accordance with published, recognized principles. The forces determined by such elastic analysis shall not exceed those given in Clause 13, and, if applicable, Clause 15.

16.10.4 The strength and stability of the member in the vicinity of openings may be determined on the basis of assumed locations of plastic hinges, such that the resulting force distributions satisfy equilibrium, provided that the analysis is carried out in accordance with Clause 8.5 (a), (b) and (f). However, for I-type members the width-thickness ratio of the flanges may meet the requirements of Class 2 sections, provided the webs meet the width-thickness limit of Class 1 sections.

16.11 Torsion

16.11.1 Beams and girders subjected to torsion shall have sufficient strength and rigidity to resist the torsional moment and forces in addition to other moment or forces. The connections and bracing of such members shall be adequate to transfer the reactions to the supports.

16.11.2 Members subjected to torsional deformations required to maintain compatability of the structure need not be designed to resist the associated torsional moments provided that the structure satisfies the requirements of equilibrium.

16.11.3 For all members subjected to loads causing torsion, the torsional deformations under specified loads shall be limited in accordance with the requirements of Clause 6.2.1.1.

17. Open-Web Steel Joists

17.1 Scope

Clause 17 provides requirements for the design, manufacture, transport and erection of open-web steel joists used in construction of buildings. Joists intended to act compositely with the deck shall be designed using the requirements of Clause 18 in conjunction with the requirements of this Clause. This Clause shall not be used for the design of joists not having an axis of symmetry in the plane of the joist.

17.2 Definitions

The following definitions apply to Clause 17:

Open-web joists or joists means simply supported steel trusses of relatively low mass with parallel or slightly pitched chords and triangulated web systems proportioned to span between masonry walls, or structural supporting members, or both, and provide direct support for floor or roof deck.

Open-web steel joists are flexural members whose design is governed by the loading given in Clause 17.5.1. The definition does not include primary trusses supporting joists, other secondary members, and special joists.

Special open-web steel joists or special joists means:

(a) Joists subjected to the loads stipulated in Clause 17.5.2; and

(b) Cantilever joists, continuous joists and joists having special support conditions; and

(c) Joists having other special requirements.

In general open-web steel joists and special open-web steel joists are manufactured on a production-line basis employing jigs, certain details of the members being standardized by the individual manufacturer.

Deck or decking means the structural floor or roof element spanning between adjacent joists and directly supported thereby. The terms deck and decking include cast-in-place or precast concrete slabs, profiled metal deck, wood plank or plywood and other relatively rigid elements suitable for floor or roof construction;

Tie joist means a joist which has at least one end connected to a column to facilitate erection and is designed to resist gravity loads only unless otherwise specified;

Span of an open-web steel joists means the distance centre-to-centre of joists bearings.

17.3 Materials

Steel for joists shall be of a structural quality, suitable for welding, meeting the requirements of Clause 5.1.1. Yield levels reported on mill test certificates shall not be used as the basis for design.

17.4 Drawings

17.4.1 Building Design Drawings

The building design drawings prepared by the building designer shall show:

(a) The uniformly distributed specified live and dead gravity loads, the unbalanced loading condition and the concentrated load conditions given in Clause 17.5.1 or 17.5.2 and any special loading conditions such as horizontal loads, end moments, net uplift, and allowances for mechanical equipment;

(b) Maximum joist spacing and where necessary camber, maximum joist depth and shoe depth;

(c) Where joists are not supported on steel members, maximum bearing pressures or sizes of bearing plates;

(d) Anchorage requirements in excess of the requirements of Clause 17.5.13;

(e) Bracing as may be required by Clause 17.8.1.

Note: It is recommended that the building design drawings include a note warning that attachments for mechanical, electrical and other services shall be made by using approved clamping devices or u-bolt type connectors and that no drilling or cutting shall be done unless approved by the building designer.

17.4.2 Joist Design Drawings

Joist design drawings prepared by the joist manufacturer shall show, at least, specified loading, factored member loads, material specification, member sizes, dimensions, spacers, welds, shoes, anchorages, bearings, field splices, bridging locations and camber.

17.5 Design

17.5.1 Loading for Open-Web Steel Joists

Unless otherwise specified by the building designer (in accordance with Clause 17.5.2), the factored moment and shear resistances of an open-web steel joist at every section shall be not less than the moment and shear due to the following factored load conditions, considered separately:

(a) A uniformly distributed load equal to the total dead and live load;

(b) An unbalanced load with 100 per cent of the total dead and live load on any continuous portion of the joist and 25 per cent of total dead and live loads on the remainder to produce the most critical effect on any component;

(c) A concentrated factored load applied at any panel point of 13.5 kN for floor joists for office or similar occupancy or 2 kN for roof joists.

17.5.2 Loading for Special Open-Web Steel Joists

The factored moment and shear resistances of special open-web steel joists at every section shall be not less than the moment and shear due to the loading conditions specified by the building designer in Clause 17.4.1(a) nor due to the factored dead load plus the following factored live load conditions (a), or (b) considered separately:

(a) For floor joists, an unbalanced live load applied on any continuous portion of the joist to produce the most critical effect on any component;

(b) The appropriate factored concentrated load specified by the Regulatory Authority; applied at any one panel point to produce the most critical effect on any component.

17.5.3 Design Assumptions

Open-web steel joists shall be designed for loads acting in the plane of the joist applied to the compression chord which is assumed to be prevented from lateral buckling by the deck.

For the purpose of determining axial forces in all members the loads may be replaced by statically equivalent loads applied at the panel points.

17.5.4 Verification of Joist Manufacturer's Design

When the adequacy of the design of a joist cannot be readily demonstrated by a rational analysis based on accepted theory and engineering practice, the joist manufacturer may elect to verify the design by test. The test shall be carried out to the satisfaction of the building designer. The test loading shall be 1.10/0.9 times the factored loads used in the design.

17.5.5 Member and Connection Resistance

Member and connection resistance shall be calculated in accordance with the requirements of Clause 13 except as otherwise specified in Clause 17.

17.5.6 Width-Thickness Ratios

17.5.6.1 General

Width-thickness ratios of compression elements of hot formed sections shall be governed by Clause 11. Width thickness ratios of compression elements of cold formed sections shall be governed by Clause 14.

17.5.6.2 **Compression Elements Supported Along One Edge**
For purposes of determining the appropriate width-thickness ratio, any stiffening effect of the deck or the joist web shall be neglected.

17.5.7 **Tension Chord**
The tension chord shall be continuous and may be designed as an axially loaded tension member unless subject to eccentricities in excess of those permitted under Clause 17.5.11.4 or to applied load between panel points. The governing radius of gyration of the tension chord or any component thereof shall be not less than 1/240 of the corresponding unsupported length. For joists with the web in the y-plane the unsupported length of chord for computing L_x/r_x shall be taken as the panel length centre-to-centre of panel points and the unsupported length of chord for computing L_y/r_y shall be taken as the distance between bridging lines connected to the tension chord. Joist shoes, when anchored, may be assumed to be equivalent to bridging lines. When net uplift is specified, the tension chord shall be designed for the resulting load reversal. Where shown on the drawings, bottom chords of joists shall be designed for end moments. Moments due to concentrated loads shall be included in the design.

17.5.8 **Compression Chord**

17.5.8.1 The compression chord shall be continuous and may be designed for axial compressive force alone when the panel length does not exceed 610 mm, when concentrated loads are not applied between the panel points, and when not subjected to eccentricities in excess of those permitted under Clause 17.5.11.4. When the panel length exceeds 610 mm the compression chord shall be designed as a continuous member subject to combined axial and bending forces.

17.5.8.2 The slenderness ratio (KL/r) of the compression chord, or of its components, shall not exceed 90 for interior panels nor 120 for end panels where the governing (KL/r) shall be the maximum value determined by the following:

(a) For x-x (horizontal) axis, L_x shall be the distance centre-to-centre of panel points. K = 0.9;

(b)For y-y (vertical) axis, L_y shall be the distance centre-to-centre of the attachments of the deck.

The spacing of attachments shall be not more than the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis nor more than 1000 mm. K = 1.0;

(c) For z-z (skew) axis of individual components, L_z shall be the distance centre-to-centre of panel points or spacers, or both. Decking shall not be considered to fulfil the function of batten plates or spacers for top chords consisting of two separated components. K = 0.9.

where

r = the appropriate radius of gyration.

17.5.8.3 Compression chords of joists in panel lengths exceeding 610 mm shall be proportioned such that:

$$\frac{C_f}{C_r} + \frac{M_f}{M_r} \leq 1.0$$

where

M_r is given in Clause 13.5 and C_r is given in Clause 13.3.

At the panel point C_r may be taken as $\phi A F_y$ and Clause 13.5(a) may be used to determine M_r provided that the chord meets the requirements of a Class 2 section and $M_f/M_p < 0.25$.

The chord shall be assumed to be pinned at the joist supports.

17.5.9 Webs

17.5.9.1 Webs shall be designed in accordance with the requirements of Clause 13 to resist the shear at any point due to the factored loads given in Clause 17.5.1 or 17.5.2. Particular attention shall be paid to possible reversals of shear.

17.5.9.2 The length of a web member shall be taken as the distance between the intersections of the axes of the web and the chords. For buckling in the plane of the web the effective length factor shall be taken as 0.9 if the web consists of individual members. For all other cases the effective length factor shall be taken as 1.0.

17.5.9.3 Web Members in Tension

The slenderness ratio of a web member in tension need not be limited.

17.5.9.4 **Web Members in Compression**
The slenderness ratio of a web member in compression shall not exceed 200.

17.5.10 **Spacers and Battens**
Compression members, consisting of two or more sections, shall be interconnected so that the slenderness ratio of each section computed using its least radius of gyration is less than or equal to the design slenderness ratio of the built-up member. Spacers or battens shall be an integral part of the joist.

17.5.11 **Connections and Splices**

17.5.11.1 Component members of joists shall be connected by welding, bolting or other approved means.

17.5.11.2 Connections and splices shall develop the factored loads required by this Standard without exceeding the factored member resistances given in Clause 17. Groove-welded splices shall develop the factored tensile resistance, T_r of the member.

17.5.11.3 Splices may occur at any point in chord or web members.

17.5.11.4 **Eccentricity Limits**
Members connected at a joint preferably shall have their gravity axes meet at a point. Where this is impractical and eccentricities are introduced such eccentricities may be neglected if they do not exceed:

(a) **For continuous web members** - The greater of the two distances measured from the neutral axis of the chord member to the extreme fibres of the chord member.

(b) **For non-continuous web members** - The distance measured from the neutral axis to the back (outside face) of the chord member.

When the eccentricity exceeds these limits, provision shall be made for the effects of total eccentricity.

Eccentricities assumed in design shall be those at maximum fabrication tolerances which shall be stated on the shop drawings.

17.5.12 **Bearings**

17.5.12.1 Bearings at ends of joists shall be proportioned so that the factored bearing resistance of the supporting material is not exceeded.

17.5.12.2 Where a joist bears, with or without a bearing plate on solid masonry or concrete support, the end of the bearing shall extend at least 90 mm beyond the face of support.

17.5.12.3 Where a joist bears on a member of the structural steel frame, the end of the bearing shall extend at least 65 mm beyond the face of the support except that when the available bearing area is restricted, this distance may be reduced provided that the bearing is adequately anchored to the support and the factored bearing resistance is not exceeded.

17.5.12.4 The bearing detail and the end panels of the joist shall be proportioned to include the effect of the eccentricity between the centre of bearing and the intersection of the axes of the chord and the end diagonal.

17.5.13 **Anchorage**

17.5.13.1 Joist ends shall be properly anchored to withstand the effect of factored loads:

(a) In no case shall the anchorage to masonry be less than:

(i) For floor joists, a 10 mm diameter rod at least 300 mm long embedded horizontally;

(ii) For roof joists, a 20 mm diameter anchor bolt 300 mm long embedded vertically with a 50 mm - 90° hook;

(b)The anchorage to steel shall be a connection capable of withstanding a horizontal load not less than 10 percent of the end reaction of the joist but not less than one 20 mm diameter bolt or a pair of fillet welds satisfying the minimum size and length requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

17.5.13.2 **Tie Joists**
Tie joists may have their top and bottom chords connected to a column. Unless otherwise specified, tie joists shall have top and bottom chord connections each at least equivalent to those required by Clause 17.5.13.1. Either top or bottom

connection shall be by means of a mechanical fastener.

17.5.13.3 Frame Action
Where joists are used as a part of a frame, the joist to column connection shall be designed to carry the moments and forces due to the factored loads (see Clause 7.2).

17.5.14 Deflection

17.5.14.1 General
Steel joists shall be proportioned so that deflection due to specified loads is within acceptable limits for the nature of the materials to be supported and the intended use and occupancy. Such deflection limits shall be as given in Clause 6.2 unless otherwise specified by the building designer.

17.5.14.2 Deflection Calculations
The deflection may be established by test or may be computed assuming a moment of inertia equal to the gross moment of inertia of the chords about the centroidal axis of the joist and multiplying the calculated deflection derived on this basis by 1.10.

17.5.15 Camber
Unless otherwise specified by the building designer the nominal camber in millimetres shall be equal to 0.07 times the square of the span expressed in metres. For tolerances see Clause 17.10.9.

17.5.16 Vibration
The building designer shall give special consideration to floor systems where unacceptable vibration may occur. The joist manufacturer when requested shall supply joist properties and details to the building designer. (See Appendix F.)

17.5.17 Welding

17.5.17.1 Arc Welding
Arc welding design and practice shall conform to CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

17.5.17.2 Resistance Welding
The resistance of resistance welded joints shall be taken as established in CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, and the related welding practice shall be in conformance with welding standards approved by the

Canadian Welding Bureau under the same CSA Standard.

17.5.17.3 **Fabricator and Erector Qualification**
Fabricators and erectors of welded construction covered by this Standard shall be certified by the Canadian Welding Bureau in Division 1 or 2.1 to the requirements of CSA Standard W47.1, Certification of Companies for Fusion Welding of Steel Structures, or CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, or both, as applicable.

17.5.17.4 The factored resistances of welds shall be those given in Table 5(b).

17.5.17.5 **Field Welding**
To achieve an adequate weldment when field welding joists to supporting members, surfaces to be welded shall be free of detrimental coatings.

17.5.17.6 **Removal of Flux and Slag**
Flux and slag shall be removed from all welds.

17.6 **Stability During Construction**
Means shall be provided to support joist chords against lateral movement and to hold the joist in the vertical or specified plane during construction.

17.7 **Bridging**

17.7.1 **General**
Bridging transverse to the span of joists may be used to meet the requirements of Clause 17.6 and also to meet the slenderness ratio requirements for chords. Bridging is not to be considered "bracing" as defined under Clause 20.3.1.

17.7.2 **Installation**
All bridging and bridging anchors shall be completely installed before any construction loads are placed on the joists except for the weight of the workmen necessary to install the bridging.

17.7.3 **Types**
Unless otherwise specified or approved by the building designer the joist manufacturer shall supply bridging which may be either the diagonal or horizontal type.

17.7.4 **Diagonal Bridging**
Diagonal bridging consisting of crossed members running from top chord to bottom chord of adjacent joists shall have a slenderness ratio (L/r) of not more than 200 where "L" is the length of the

diagonal bridging member, or one-half this length when crossed members are connected at their point of intersection, and "r" is the least radius of gyration. All diagonal bridging shall be connected adequately to the joists by bolts or welds.

17.7.5 **Horizontal Bridging**
A line of horizontal bridging shall consist of a continuous member attached to either the top chord or the bottom chord. Horizontal bridging members shall have a slenderness ratio of not more than 300.

17.7.6 **Attachment of Bridging**
Attachment of diagonal and horizontal bridging to joist chords shall be by welding or mechanical means capable of resisting an axial load of at least 3 kN in the attached bridging member. These welds should meet the minimum length requirements stipulated in CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

17.7.7 **Anchorage of Bridging**
Each line of bridging shall be adequately anchored at each end to sturdy walls or to main components of the structural frame, if practicable. If not practicable, diagonal and horizontal bridging shall be provided in combination between adjacent joists near the ends of bridging lines.

The ends of joists designed to bear on their bottom chords shall be held adequately in position by attachments to the walls or to the structural frame or by lines of bridging located at the ends except where such ends are built into masonry or concrete walls.

17.7.8 **Bridging Systems**
Bridging systems, including sizes of bridging members, and all necessary details, shall be shown on the erection diagrams. If a specific bridging system is required by the design, the design drawings shall show all information necessary for the preparation of shop details and erection diagrams.

17.7.9 **Spacing of Bridging**
Diagonal and horizontal bridging, whichever is furnished, shall be spaced so that the unsupported length of the chord between bridging lines, or between laterally supported ends of the joist and adjacent bridging lines, does not exceed:

(a) For compression chords, 170r;

(b) For tension chords, 240r;

where

r = the applicable chord radius of gyration about its axis in the plane of the web

Ends of joists anchored to supports may be assumed to be equivalent to bridging lines. If not so anchored before installing deck, the distance from the face of the support to the nearest bridging member in the plane of the bottom chord shall not exceed 120r. In no case shall there be less than one line of horizontal or diagonal bridging attached to each joist spanning 4000 mm or more. If only a single line of bridging is required, it shall be placed at the centre of the joist span. If bridging is not used on joists less than 4000 mm in span, the ends of such joists shall be anchored to the supports so as to prevent overturning of the joist during placement of the deck.

17.8 Decking

17.8.1 Decking to Provide Lateral Support

Decking shall bear directly on the top chord of the joist and shall be sufficiently rigid to provide lateral support to the compression chord of the joist. In special cases where the decking is incapable of furnishing the required lateral support, the compression chord of the joist shall be braced laterally in accordance with the requirements of Clause 20.3.

17.8.2 Attachments of decking considered to provide lateral support shall be capable of staying the top chords laterally. Attachments shall be deemed to fulfil this requirement when the attachments as a whole are adequate to resist a force in the plane of the decking of not less than 5 per cent of the maximum force in the top chord and assumed to be uniformly distributed along the length of the top chord. The spacing of attachments shall be not more than the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis nor more than 1000 mm.

17.8.3 Diaphragm Action

Where decking is used in combination with joists to form a diaphragm for the purpose of transferring lateral applied loads to vertical bracing systems, special attachment requirements shall be fully specified on the building design drawings.

17.8.4 Cast-in-place slabs used as decking shall have a minimum thickness of 50 mm. Forms for cast-in-place slabs shall not cause lateral displacement of the top chords of joists during installation of the forms or the placing of the concrete. Non-removable forms shall be positively attached to top chords by means of clips, ties, wedges, fasteners, or other suitable means at intervals not exceeding 1000 mm; however, there shall be at least two attachments in the width of each form at each joist. Forms and their method of attachment shall be such that the cast-in-place slab, after hardening, is capable of furnishing lateral support to the joist chords.

17.9 Shop Painting

Joists shall have one shop coat of protective paint of a type standard with the manufacturer unless otherwise specified.

17.10 Manufacturing Tolerances

17.10.1 The tolerance on the specified depth of the manufactured joist shall be ± 7 mm.

17.10.2 The maximum deviation from the design location of a panel point measured along the length of a chord shall be 13 mm. In joists in which an individual end diagonal is attached to the bottom chord or in which the end diagonal is a continuation of an upturned bottom chord the gravity axes of the members in such a joint should meet at a point. (See Clause 17.5.11.4.)

17.10.3 The maximum deviation from the design location of a panel point measured perpendicular to the longitudinal axis of the chord and in the plane of the joist shall be 7 mm.

17.10.4 The connections of web members to chords shall not deviate laterally more than 3 mm from that assumed in the design.

17.10.5 The maximum sweep of a joist or any portion of the length of the joist upon completion of manufacture shall be 1/500 of the length on which the sweep is measured.

17.10.6 The maximum tilt of bearing shoes shall be 1 in 50 measured from a plane perpendicular to the plane of the web and parallel to the longitudinal axis of the joist.

17.10.7 The tolerance on the specified shoe depth shall be ± 3 mm.

17.10.8 The tolerance on the specified length of the joist shall be ± 7 mm. The connection holes in a joist shall not vary from the detailed location by more than 2 mm for members 10 000 mm or less in length or by more than 3 mm for members over 10 000 mm in length.

17.10.9 The tolerance on the nominal or specified camber shall be

$$\pm\left(6 \text{ mm} + \frac{\text{Span, in mm}}{4\ 000}\right)$$

The resulting actual minimum camber in a joist is to be +3 mm except that the maximum range in camber for joists of the same span shall be limited to 20 mm.

17.11 Inspection and Quality Control

17.11.1 Inspection

Material and workmanship at all times shall be accessible for inspection by qualified inspectors representing the building designer. Random in-process inspection shall be carried out by the manufacturer and all joists shall be thoroughly inspected by the manufacturer before shipping.

17.11.2 Identification and Control of Steel

Steel used in the manufacture of joists shall at all times, in the manufacturers' plant be marked to identify its specification (and grade, where applicable). This shall be done by suitable markings or by recognized colour coding or by any system devised by the manufacturer that will ensure to the satisfaction of the building designer that the correct material is being used.

17.11.3 Quality Control

Upon request of the building designer the manufacturer shall provide evidence of having suitable quality control measures to ensure that the joists meet all requirements specified. When testing is part of the manufacturer's normal quality control program, the loading criterion shall be 1.0/0.9 times the factored loads for the materials used in the joists.

For resistance welding and quality control procedures outlined in CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, shall be met. For arc-welding quality control, the requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding), shall be met.

17.12 **Handling and Erection**

17.12.1 **General**

Care shall be exercised to avoid damage during strapping, transport, unloading, site storage and piling, and erection. Dropping of joists shall not be permitted. Special precautions shall be taken when erecting long, slender joists and preferably hoisting cables shall not be released until the member is stayed laterally by at least one line of bridging. Joists shall have all bridging attached and be permanently fastened into place before the application of any loads. Heavy construction loads shall be adequately distributed so as not to exceed the capacity of any joist. Field welding shall not cause damage to joists, bridging, deck and supporting steel members.

17.12.2 **Erection Tolerances**

17.12.2.1 The maximum sweep of a joist or a portion of the length of a joist upon completion of erection shall not exceed the requirements of Clause 17.10.5, and shall be in accordance with the general requirements of Clause 29.

17.12.2.2 All members shall be free from twists, sharp kinks and bends.

17.12.2.3 **Location of Joist**

When joists are finally fastened in position in the field, the maximum deviation from the location shown on the erection drawings shall be 15 mm.

17.12.2.4 The deviation, normal to the specified plane of the web of a joist, shall not exceed 1/50 of the depth of the joist.

18. **Composite Beams and Columns**

18.1 **Application**

18.1.1 The provisions of Clause 18 apply to:

(a) Composite beams consisting of steel sections, trusses or joists interconnected with either a reinforced concrete slab or a steel deck with concrete cover slab;

(b) Composite columns consisting of steel hollow structural sections completely filled with concrete.

18.1.2 For any requirement not covered in Clause 18 the design shall conform to the provisions of this Standard.

18.2 **Definitions**

The following definitions apply to Clause 18.

Steel deck means a load-carrying steel deck consisting of either:

(a) A single fluted element (non-cellular deck) ; or

(b) A two element section comprising a fluted element in conjunction with a flat sheet (cellular deck).

The maximum depth of the deck shall be 80 mm and the average width of the minimum flute shall be 50 mm. Steel deck may be of a type intended to act compositely with the cover slab in supporting applied load;

Flute means that portion of the steel deck which forms a valley.

Concrete means Portland cement concrete in accordance with CSA Standard CAN3-A23.1, Concrete Materials and Methods of Concrete Construction;

Rib means that portion of the concrete slab which is formed by the steel deck flute.

Slab means a reinforced cast-in-place concrete slab at least 65 mm in effective thickness. The area equal to the design effective width times effective slab thickness shall be free of voids or hollows except for those specifically permitted in the definition of effective slab thickness;

Cover slab means the concrete above the flutes of steel deck. All flutes shall be filled with concrete so as to form a rib slab;

Effective cover slab thickness, t, means the minimum thickness of concrete measured from the top of the cover slab to the top of the steel deck. This thickness shall be not less than 65 mm unless the adequacy of a lesser thickness has been established by appropriate tests;

Effective slab thickness, t, means the overall slab thickness, provided that;

(a) The slab is cast with a flat underside; or

(b) The slab is cast on corrugated steel forms having a height of corrugation not greater than 0.25 times the overall slab thickness; or

(c) The slab is cast on fluted steel forms whose profile meets the following requirements. The minimum concrete rib width shall be 125 mm; the maximum rib height shall be 40 mm but not more than 0.4 times the overall slab thickness; the average width between ribs shall not exceed 0.25 times the overall slab thickness nor 0.2 times the minimum width of concrete.

In all other cases, effective slab thickness means the overall slab thickness minus the height of form flute or corrugation;

Steel joist means an open web steel joist suitable for composite design;

Steel section means a steel structural section with a solid web, or webs, suitable for composite design. Web openings are permissible only on condition that their effects are fully investigated and accounted for in the design.

18.3 Composite Beams

18.3.1 General

18.3.1.1 Calculation of deflections shall take into account the effects of creep of concrete, shrinkage of concrete, and increased flexibility resulting from partial shear connection and from interfacial slip. These effects shall be established by test or analysis, where practicable. Consideration shall also be given to effects of full or partial continuity in the steel beams and concrete slabs in reducing calculated deflections.

In lieu of tests or analysis the effects of partial shear connection, interfacial slip, creep, and shrinkage may be assessed as follows:

(a) increased flexibility resulting from partial shear connection and interfacial slip: calculate the elastic deflections using an effective moment of inertia,

$$I_e = I_s + 0.85\ p\ (I_t - I_s)$$

where

I_s = moment of inertia of the steel section

I_t = moment of inertia of the transformed composite section
p = fraction of full shear connection expressed as a decimal

(b) creep: increase elastic deflections due to dead loads and long-term live loads as computed in (a), by 15%.

(c) shrinkage of concrete: calculate deflections using a selected shrinkage strain assuming the composite beam is bent in single curvature by a constant moment. The shrinkage strain is affected by such factors as age of concrete, ratio of slab volume to surface area, concrete properties (water/cement ratio, per cent fines, entrained air, and cement content), and the restraint provided by steel beam and deck. See Appendix K for methods of computing deflections due to shrinkage strains.

18.3.1.2 The web area of steel sections or web system of steel trusses and joists shall be proportioned to carry the total vertical shear V_f.

18.3.1.3 End connections of steel sections, trusses and joists shall be proportioned to transmit the total end reaction of the composite beam.

18.3.2 Design Effective Width of Concrete

18.3.2.1 Slabs or cover slabs extending on both sides of the steel section or joist shall be deemed to have a design effective width, b, equal to the least of:

(a) 0.25 times the composite beam span;

(b) 16 times the overall slab thickness (thickness of cover slab plus steel deck depth), plus the width of the top flange of the steel section or top chord of the steel truss or joist;

(c) The average distance from the centre of the steel section, truss or joist to the centres of adjacent parallel supports.

18.3.2.2 Slabs or cover slabs extending on one side only of the supporting section or joist shall be deemed to have a design effective width, b, not greater than the width of top flange of the steel section, or top chord of the steel joist, plus the least of:

(a) 0.1 times the composite beam span;

(b) 6 times the overall slab thickness (thickness of cover slab plus steel deck depth);

(c) 0.5 times the clear distance between the steel section or joist and the adjacent parallel support.

18.3.3 Slab Reinforcement

Slabs shall be adequately reinforced to support all specified loads and to control cracking both parallel and transverse to the composite beam span. Reinforcement parallel to the span of the beam in regions of negative bending moment of the composite beam shall be anchored by embedment in concrete which is in compression. The reinforcement of slabs which are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention.

The possibility of longitudinal cracking due to composite action, directly over the steel section or joist, shall be controlled by the provision of additional transverse reinforcement or other effective means unless it is known from experience that cracking due to composite action is unlikely. Such additional reinforcement shall be placed in the lower part of the slab and anchored so as to develop the yield strength of the reinforcement. The area of such reinforcement shall be not less than 0.005 times the concrete area in the longitudinal direction of the beam and shall be uniformly spaced along the composite beam span.

18.3.4 Composite Action With Steel Deck

Cover slabs intended to act compositely with steel deck shall have reinforcement transverse to the span of the composite beam as required. Reinforcement shall be not less than that required by the specified fire resistance design of the assembly.

18.3.5 Interconnection

18.3.5.1 Except as permitted by Clauses 18.3.5.2 and 18.3.5.4 interconnection between steel sections, trusses or joists and slabs or cellular steel deck with cover slabs shall be attained by the use of shear connectors as prescribed in Clause 18.3.6.

18.3.5.2 Unpainted steel sections, trusses or joists supporting slabs and totally encased in concrete do not require interconnection by means of shear connectors provided that:

(a) A minimum of 50 mm of concrete covers all portions of the steel section or joist, except as noted in Item (c);

(b) The cover in Item (a) is reinforced to prevent spalling; and

(c) The top of the steel section or joist is at least 40 mm below the top and 50 mm above the bottom of the slab.

18.3.5.3 Studs may be welded through a maximum of two steel sheets in contact, each not more than 1.71 mm in overall thickness including coatings (1.52 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise holes for placing studs shall be made through the sheets as necessary. Welded studs shall meet the requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

18.3.5.4 Other methods of interconnection which have been adequately demonstrated by test and verified by analysis may be used to effect the transfer of forces between the steel section, truss or joist and the slab or steel deck with cover slab. In such cases the design of the composite member shall conform to the design of a similar member employing shear connectors, insofar as practicable.

18.3.5.5 The diameter of a welded stud shall not exceed 2.5 times the thickness of the part to which it is welded, unless test data satisfactory to the designer is provided to establish the capacity of the stud as a shear connector.

18.3.6 **Shear Connectors**

The factored shear resistance q_r, of a shear connector shall be established by tests acceptable to the designer, except that the following values shall be acceptable without further verification and where ϕ_{sc}, the resistance factor for shear connectors, is taken as 0.80:

(a) **End welded studs, headed or hooked with h/d > 4**

$$q_r = 0.5\ \phi_{sc} A_{sc} \sqrt{f'_c E_c} < \phi_{sc} F_u^{*} A_{sc} \quad \text{(newtons)}$$

* F_u for commonly available studs is 415 MPa.

This value is limited to designs incorporating a solid concrete slab; or designs incorporating a ribbed slab formed by casting a concrete cover slab on a fluted steel deck in which the rib average width is at least twice the height of the formed concrete rib and the projection of the stud, based on its length prior to welding, is at least two stud diameters above the top surface of the steel deck;

(b) **End welded studs, headed, in selected cases** - Table 13 gives values of q_r for selected cases of composite beams incorporating steel deck not covered by Item (a). Cover slabs shall consist of normal density concrete (2300 kg/m^3) with $f'_c > 20$ MPa.

Table 13
Shear Capacities of Studs for Selected Cases

Height of Cellular Steel Deck (mm)	Average Rib Width, Minimum (mm)	Depth of Cover Slab (mm)	Stud Size d x h (mm x mm)	No. of Studs per Rib	Shear Capacity* q_r (newtons)
38/43	50	65	14 x 75	1	24 000ϕ_{sc}
38/43	50	65	20 x 75	1	50 000ϕ_{sc}
				2	76 000ϕ_{sc} per pair
38/43	50	90	20 x 100	1	79 000ϕ_{sc}
				2	113 000ϕ_{sc} per pair

* Shear capacities given in Table 13 are derived from test and reflect the influence of rib geometry and stiffness on the useful capacity of the studs.

(c) **Channel connectors**

$$q_r = 36.5\ \phi_{sc}\ (t_f + 0.5t_w)L_c\sqrt{f'_c} \quad \text{(newtons)}$$

This formula is limited to designs incorporating a solid concrete slab of normal density concrete (2300 kg/m^3) with $f'_c > 20$ MPa.

18.3.7 **Ties**
Mechanical ties shall be provided between the steel section, truss or joist and the slab or steel deck to prevent separation. Shear connectors may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1000 mm and the average spacing in a span should not exceed 600 mm nor be greater than that required to achieve any specified fire resistance rating of the composite assembly.

18.4 Design of Composite Beams With Shear Connectors

18.4.1 The composite beam shall consist of steel section, truss or joist, shear connectors, ties, and slab or steel deck with cover slab.

18.4.2 The properties of the composite section shall be computed neglecting any concrete area in tension within the maximum effective area equal to effective width times effective thickness. If a steel truss or joist is used the area of its top chord shall be neglected in determining the properties of composite section and only Clause 18.4.3(a) is applicable.

18.4.3 The factored moment resistance, M_{rc}, of the composite section with the slab or cover slab in compression shall be computed as follows, where $\phi = 0.90$ and ϕ_c, the resistance factor for concrete = 0.60:

(a) **Case 1** - Full shear connection and plastic neutral axis is in the slab that is $Q_r > \phi A_s F_y$ and

$\phi A_s F_y < 0.85\phi_c btf'_c$ where Q_r equals the sum of the

factored resistances of all shear connectors between points of maximum and zero moment.

$$M_{rc} = T_r e' = \phi A_s F_y e'$$

where e' = the lever arm and is computed using

$$a = \frac{\phi A_s F_y}{0.85\phi_c bf'_c}$$

(b) **Case 2** -Full shear connection and plastic neutral axis in the steel section that is $Q_r > 0.85\phi_c btf'_c$ and $0.85\phi_c btf'_c < \phi A_s F_y$

$$M_{rc} = C_r e + C'_r e'$$

$$C'_r = 0.85\phi_c btf'_c$$

$$C_r = \frac{\phi A_s F_y - C'_r}{2}$$

(c) **Case 3** - Partial shear connection that is $Q_r < 0.85\phi btf'_c$ and $<\phi A_s F_y$

$$M_{rc} = C_r e + C'_r e'$$

$$C'_r = \phi_c Q_r$$

$$C_r = \frac{\phi A_s F_y - C'_r}{2}$$

where e' = the lever arm and is computed using

$$A = \frac{C'_r}{0.85\phi_c bf'_c}$$

18.4.4 No composite action shall be assumed in computing flexural strength when Q_r is less than 0.5 times the lesser of $0.85\phi btf'_c$ and $\phi A_s F_y$. No composite action

shall be assumed in computing deflections when Q_r is less than 0.25 times the lesser of $0.85\phi_c btf'_c$ and $\phi A_s F_y$.

18.4.5 For full shear connection, the total horizontal shear, V_h, at the junction of the steel section or joist and the concrete slab or steel deck, to be resisted by shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment, shall be:

$$V_h = \phi A_s F_y$$

for Case 1 and

$$V_h = 0.85\phi_c btf'_c$$

for Case 2 as defined in Clause 18.4.3(a) and (b), where $Q_r > V_h$.

18.4.6 For partial shear connection, the total horizontal shear as defined in Clause 18.4.3(c) shall be:

$$V_h = Q_r$$

18.4.7 Composite beams employing steel sections and concrete slabs may be designed as continuous members. The factored moment resistance of the composite section, with the concrete slab in the tension area of the composite section shall be the factored moment resistance of the steel section alone except that when sufficient shear connectors are placed in the negative moment region, suitably anchored concrete slab reinforcement parallel to the steel sections and within the design effective width of the concrete slab may be included in computing the properties of the composite section. The total horizontal shear, V_h, to be resisted by shear connectors between the point of maximum negative bending moment and the adjacent point of zero moment shall be taken as:

$$V_h = \phi A_r F_{yr}$$

18.4.8 The number of shear connectors to be located each side of the point of maximum bending moment (positive or negative, as applicable) and distributed between that point and the adjacent point of zero moment shall be not less than:

$$n = \frac{V_h}{q_r}$$

Shear connectors may be spaced uniformly except that in a region of positive bending, the number of shear connectors required between any concentrated load applied in that region and the nearest point of zero moment shall be not less than n"

$$n" = n\left(\frac{M_{f1} - M_r}{M_f - M_r}\right)$$

where

M_{f1} = positive bending moment at concentrated load point
M_r = moment resistance of the steel section alone
M_f = maximum positive bending moment

18.5 Design of Composite Beams Without Shear Connectors

18.5.1 Unpainted steel sections or joists supporting concrete slabs and encased in concrete in accordance with Clause 18.3.5.2 may be proportioned on the basis that the composite section supports the total load.

18.5.2 The properties of the composite section for determination of load-carrying capacity shall be computed by ultimate strength methods, neglecting any area of concrete in tension.

18.5.3 As an alternative method of design, encased simple span steel sections or joists may be proportioned on the basis that the steel section or joist along supports 0.90 times the total load.

18.6 Unshored Beams

For composite beams, unshored during construction, the stresses in the tension flange of the steel section or joist, due to the loads applied before the concrete strength reaches $0.75f_c'$ plus the stresses at the same location, due to the remaining specified loads considered to act on the composite section shall not exceed $0.90F_y$.

18.7 Beams During Construction

The steel section or joist alone shall be proportioned to support all factored loads applied prior to hardening of the concrete without exceeding its calculated capacity under the conditions of lateral support or shoring, or both, to be furnished during construction.

$M_f \leq M_r$

18.8 Design of Composite Columns (Concrete-Filled Hollow Structural Sections)

18.8.1 Hollow structural sections designated as Class 1, 2 or 3 sections which are completely filled with concrete may be assumed to carry compressive load as composite columns. Class 4 hollow structural sections completely filled with concrete may also be designated as composite columns providing the width-thickness ratios of the walls of rectangular sections do not exceed $1350/\sqrt{F_y}$, and the outside diameter to thickness ratio of circular sections do not exceed $28\ 000/F_y$.

18.8.2 The proportion of the axial load assumed to be carried by the concrete shall be applied by direct bearing on the concrete or, alternatively, other methods of load application to the concrete may be employed if their adequacy has been demonstrated by test.

18.8.3 The factored compressive resistance of a composite column shall be taken as:

$$C_{rc} = \tau C_r + \tau' C_r'$$

where

$$C_r' = 0.85\phi_c f_c' A_c \lambda^{-2}[\sqrt{1 + 0.25\lambda_c^{-4}} - 0.5\lambda_c^{-2}]$$

in which $\lambda_c = \dfrac{KL}{r_c}\sqrt{\dfrac{f_c'}{\pi^2 E_c}}$

r_c = radius of gyration of the concrete area, A_c,

E_c = initial elastic modulus for concrete in MPa, considering the effects of long term loading. For normal weight concrete, with f_c' expressed in MPa, this may be taken as:

$$(1 + S/T)\ 2500\ \sqrt{f_c'}$$

where S is the short term load and T is the total load on the column.

For all rectangular hollow structural sections, and for circular hollow structural sections with height to diameter ratio of 25 or greater, $\tau = \tau' = 1.0$

Otherwise $$\tau = \frac{1}{\sqrt{1 + \rho + \rho^2}}$$

and $$\tau' = 1 + \left(\frac{25\rho^2\tau}{(D/t)}\right)\left(\frac{F_y}{0.85f'_c}\right)$$

where $\rho = 0.02(25 - L/D)$

18.8.4 Where bending as well as axial compression is to be resisted, the bending shall be assumed to be resisted by the steel section alone. The steel section shall be proportioned as a beam-column to carry the total bending, plus axial compression equal to the difference between the total axial compression and that portion which can be sustained by the concrete so that:

(a) $$\frac{M_f}{\tau M_r} \leq 1.0$$

(b) $$\frac{C_f - \tau' C'_r}{\tau_{cr}} + \frac{\omega M_f}{\tau M_r\left(1 - \left(\frac{C_f - \tau' C'_r}{C_e}\right)\right)} \leq 1.0$$

when $C_f > \tau' C'_r$

19. General Requirements for Built-Up Members

19.1 General Requirements for Compression Members

19.1.1 All components of built-up compression members and the transverse spacing of their lines of connecting bolts or welds shall meet the requirements of Clause 10 and 11.

19.1.2 All component parts in contact with one another at the ends of built-up compression members shall be connected by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1.5 times the width of the member, or by continuous welds having a length not less than the width of the member.

19.1.3 Unless closer spacing is required for transfer of load, or for sealing inaccessible surfaces, the longitudinal spacing, in line, between intermediate bolts or clear longitudinal spacing between intermittent welds in built-up compression members shall not exceed the following, as applicable:

(a) For compression members composed of two or more rolled shapes in contact or separated from one another by intermittent fillers, the slenderness ratio of any shape between points of interconnection shall not exceed the slenderness ratio of the built-up member. The least radius of gyration of each component part shall be used in computing the slenderness ratio of that part between points of interconnection with other component parts;

(b) $330t/\sqrt{F_y}$ but not more than 300 mm for the outside component of the section consisting of a plate when the bolts on all gauge lines or intermittent welds along the component edges are not staggered, where t = thickness of outside plate;

(c) $525t/\sqrt{F_y}$ but not more than 450 mm for the outside component of the section consisting of a plate when the bolts or intermittent welds are staggered on adjacent lines, where t = thickness of outside plate.

19.1.4 The spacing requirements of Clauses 19.1.3, 19.2.3 and 19.2.4 will not always provide a continuous tight fit between components in contact. When the environment is such that corrosion could be a serious problem, the spacing of bolts or welds may need to be less than the specified maximum.

19.1.5 Open sides of compression members built up from plates or shapes shall be connected to each other by lacing, batten plates, or perforated cover plates.

19.1.6 Lacing shall provide a complete triangulated shear system and may consist of bars, rods or shapes. The spacing of connections of lacing to a main component shall be such that the slenderness ratio of a main component between these points of connection does not exceed the governing slenderness ratio of the member as a whole. Lacing shall be proportioned to resist a shear, normal to the longitudinal axis of the member, of not less than 2.5 per cent of the total axial load on the member plus the shear from transverse loads, if any.

19.1.7 The slenderness ratio of lacing members shall not exceed 140. The effective length for single lacing shall be the distance between connections to the main components; for double lacing connected at the intersections, the effective length shall be 70 per cent of that distance.

19.1.8 Lacing members shall preferably be inclined to the longitudinal axis of the built-up member at an angle of not less than 45°.

19.1.9 Lacing systems shall have diaphragms in the plane of the lacing and as near the ends as practicable and at intermediate points where lacing is interrupted. Such diaphragms may be plates (tie plates) or shapes.

19.1.10 End tie plates used as diaphragms shall have a length not less than the distance between the lines of bolts or welds connecting them to the main components of the member. Intermediate tie plates shall have a length not less than one-half that prescribed for end tie plates. The thickness of tie plates shall be at least 1/60 of the width between lines of bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or clear longitudinal spacing between welds shall not exceed 150 mm. At least three bolts shall connect the tie plate to each main component, or, alternatively, a total length of weld not less than one-third the length of tie plate shall be used.

19.1.11 Shapes used as diaphragms shall be proportioned and connected to transmit from one main component to the other a longitudinal shear equal to 5 per cent of the axial compression in the member.

19.1.12 Perforated cover plates may be used in lieu of lacing and tie plates on open sides of built-up compression members. The net width of such plates at access holes shall be assumed available to resist axial load provided that:

(a) The width-thickness ratio conforms to Clause 11;

(b) The length of the access hole does not exceed twice its width;

(c) The clear distance between access holes in the direction of load is not less than the transverse distance between lines of bolts or welds connecting the perforated plate to the main components of the built-up member;

(d) The periphery of the access hole at all points has a minimum radius of 40 mm.

19.1.13 Battens consisting of plates or shapes may be used on open sides of built-up compression members which do not carry primary bending in addition to axial load. Battens shall be provided at the ends of the member, at locations where the member is laterally supported along its length and elsewhere as determined by the following spacing requirements:

(a) When the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens is equal to or less than 80 per cent of the slenderness ratio with respect to the axis parallel to the battens, the spacing between battens shall be such that the slenderness ratio of a main component between ends of adjacent batten plates shall not exceed 50, nor shall it exceed 70 per cent of the slenderness ratio of the built-up member with respect to the axis parallel to the battens;

(b) When the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens is more than 80 per cent of the slenderness ratio with respect to the axis parallel to the battens, the spacing between battens shall be such that the slenderness ratio of a main component between ends of adjacent batten plates shall not exceed 40, nor shall it exceed 60 per cent of the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens.

19.1.14 Battens shall have a length not less than the distance between lines of bolts or welds connecting them to the main components of the member and a thickness not less than 1/60 of this distance if the batten consists of a flat plate. Battens and their connections shall be proportioned to resist simultaneously a longitudinal shear force, V_f, and a moment, M_f.

where

$$V_f = \frac{0.025 C_f d}{na} \text{ (Newtons)}$$

$$M_f = \frac{0.025 C_f d}{2n} \text{ (N·mm)}$$

d = longitudinal distance centre-to-centre of battens (mm)
a = distance between lines of bolts or welds connecting the batten to each main component (mm)
n = number of parallel planes of battens

19.2 General Requirements for Tension Members

19.2.1 Tension members composed of two or more shapes, plates or bars separated from one another by intermittent fillers shall have the components interconnected at fillers spaced so that the slenderness ratio of any component between points of interconnection shall not exceed 300.

19.2.2 Tension members composed of two plate components in contact or a shape and a plate component in contact shall have the components interconnected so that the spacing between connecting bolts or clear spacing between welds does not exceed 36 times the thickness of the thinner plate nor 450 mm (see Clause 19.1.3).

19.2.3 Tension members composed of two or more shapes in contact shall have the components interconnected so that the spacing between connecting bolts or the clear spacing between welds does not exceed 600 mm, except where it can be determined that a greater spacing would not affect the satisfactory performance of the member (see Clause 19.1.3).

19.2.4 Tension members composed of two separated main components may have either perforated cover plates or tie plates on the open sides of the built-up member. Tie plates including end tie plates shall have a length not less than two-thirds of the transverse distance between bolts or welds connecting them to the main components of the member and shall be spaced so that the slenderness ratio of any component between the tie plates does not exceed 300. The thickness of tie plates shall be at least 1/60 of the transverse distance between the bolts or welds connecting them to the main components and the longitudinal spacing of the bolts or welds shall not exceed 150 mm. Perforated cover plates shall comply with the requirements of Clause 19.1.11(b), (c), and (d).

19.3 General Requirements for Open Box-Type Beams and Grillages

Where two or more rolled beams or channels are used side-by-side to form a flexural member, they shall be connected together at intervals of not more than 1500 mm. Through bolts and separators may be used, provided that in beams having a depth of 300 mm or more, no fewer than two bolts shall be used at each separator location. When concentrated loads are carried from one beam to the other, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be bolted or welded between the beams. The design of members shall provide for torsion resulting from any unequal distribution of loads. Where beams are exposed, they shall be sealed against corrosion of interior surfaces, or spaced sufficiently far apart to permit cleaning and painting.

20. Stability of Structures and Individual Members

20.1 General

20.1.1 In the design of a steel structure care shall be taken to ensure that the structural system is adequate to resist the forces caused by the factored loads and to ensure that a complete structural system is provided to transfer the factored loads to the foundations, particularly when there is a dependence on walls, floors, or roofs acting as shear resisting elements or diaphragms. (See also Clause 8.6.)

Note: The structure should also be checked to ensure that adequate resistance to torsional deformations has been provided.

20.1.2 Design drawings shall indicate all load resisting elements essential to the integrity of the completed structure and shall show details necessary to ensure the effectiveness of the load resisting system. Design drawings shall also indicate the requirements for roofs and floors used as diaphragms.

20.1.3 Erection diagrams shall indicate all load resisting elements essential to the integrity of the completed structure. Permanent and temporary load resisting elements essential to the integrity of the partially completed structure shall be clearly specified on the erection diagrams.

20.1.4 Where the portion of the structure under consideration does not provide adequate resistance to lateral forces, provision shall be made for transferring the forces to adjacent lateral load resisting elements.

20.2 Stability of Columns

Beam-to-column connections shall have adequate strength to transfer the lateral forces produced by possible out-of-plumbness as specified in Clause 29.9.1. These forces shall be computed for the loading cases of Clause 7.2.3 using the appropriate load combination factors.

20.3 Stability of Beams, Girders and Trusses

20.3.1 Bracing members assumed to provide lateral support to the compression flange of beams and girders, or to the compression chord of trusses, and the connections of such bracing members, shall be proportioned to resist a force equal to 1 per cent of the force in the compression flange or chord at the point of support.

20.3.2 When bracing of the compression flange or chord is effected by a slab or deck, the slab or deck and the means by which the computed bracing forces are transmitted between the flange or chord and the slab or deck shall be adequate to resist a force in the plane of the slab or deck. This force shall be considered to be uniformly distributed along the length of the compression flange or chord, and shall be taken as at least 5 per cent of the maximum force in the flange or chord, unless a lesser amount can be justified by analysis.

20.3.3 Consideration shall be given to the probable accumulation of forces when a bracing member must transfer forces from one braced member to another.

20.3.4 Members restraining beams and girders designed to resist loads causing torsion shall be proportioned according to the requirements of Clause 16.11. Special consideration shall be given to the connections of asymmetric section such as channels, angles and zees.

21. Connections

21.1 Alignment of Members

Axially loaded members meeting at a joint shall have their gravity axes intersect at a common point if practicable; otherwise the results of bending due to the joint eccentricity shall be provided for.

21.2 Unrestrained Members

Except as otherwise indicated on the design drawings, all connections of beams, girders, and trusses shall be designed and detailed as flexible and ordinarily may be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action at the specified load levels in the connection is permitted.

21.3 Restrained Members

When beams, girders, or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load. When beams are rigidly framed to the flange of an H-type column, stiffeners shall be provided on the column web as follows:

(a) Opposite the compression flange of beam when

$$B_r = \phi w_c (t_b + 5k) F_{yc} < \frac{M_f}{d_b}$$

except that for members with Class 3 or 4 webs,

$$B_r = \phi \frac{640\ 000}{(h_c/w_c)^2} w_c (t_b + 5k)$$

(b) Opposite the tension flange of beam when

$$T_r = \phi 7 t_c^2 F_{yc} < \frac{M_f}{d_b}$$

where

w_c = thickness of column web
t_b = thickness of beam flange
k = distance from outer face of column flange to web toe of fillet, or to web toe of flange-to-web weld in a welded column
F_{yc} = specified yield point of column
d_b = depth of beam
h_c = clear depth of column web
t_c = thickness of column flange

The stiffener or pair of stiffeners opposite either beam flange must develop a force equal to:

$$F_{st} = \frac{M_f}{d_b} - B_r$$

Stiffeners shall also be provided on the web of columns, beams or girders if V_r computed from Clause 13.4.2 is exceeded, in which case the stiffener or stiffeners must transfer a shear force equal to:

$$V_{st} = V_f - 0.55 \phi w d F_y$$

In all cases the stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one face of the column only, the stiffeners need not be longer than one-half the depth of the column.

21.4 **Connections of Tension or Compression Members**

The connections at ends of tension members or compression members not finished to bear shall develop the force due to the factored loads. However the connection shall be designed for not less than 50 per cent of the resistance of the member based on the condition (tension or compression) that governs the selection of the member.

21.5 **Bearing Joints in Compression Members**

21.5.1 Where columns bear on bearing plates, or are finished to bear at splices, there shall be sufficient fasteners or welds to hold all parts securely in place.

21.5.2 Where other compression members are finished to bear, the splice material and connecting fasteners or welds shall be arranged to hold all parts in place and shall be proportioned for 50 per cent of the computed load.

21.6 **Lamellar Tearing**

Corner or "T" joint details of rolled structural members, or plates involving transfer of tensile forces in the through-thickness direction resulting from shrinkage due to welding executed under conditions of restraint, shall be avoided where possible. If this type of connection cannot be avoided, measures shall be taken to minimize the possibility of lamellar tearing.

21.7 **Placement of Fasteners and Welds**

Except in members subjected to repeated loads (as defined in Clause 15), disposition of fillet welds to balance the forces about the neutral axis or axes for end-connections of single angle, double angle, or similar types of axially loaded members is not required. Eccentricity between the gravity axes of such members and the gauge lines of bolted end-connections also may be neglected. In axially loaded members subjected to repeated loads, the fasteners or welds in end connections shall have their centre of gravity on the gravity axis of the member unless provision is made for the effect of the resulting eccentricity.

21.8 **Fillers**

21.8.1 When load-carrying fasteners pass through fillers with a total thickness greater than 6 mm in bearing-type shear connections, the fillers shall be extended beyond the splice material and the filler extension shall be secured by sufficient fasteners to distribute the total force in the member uniformly over the combined section of the member and the filler, or alternatively an equivalent number of fasteners shall be included in the connection.

21.8.2 In welded construction, any filler with a total thickness greater than 6 mm shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler, as an eccentric load. Welds connecting the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler 6 mm or less in thickness shall have its edges made flush with the edges of the splice plate and the required weld size shall be equal to the thickness of the filler plate plus the size necessary to transmit the splice plate load.

21.9 Welds in Combination

If two or more of the general types of weld (groove, fillet, plug, or slot) are combined in a single connection, the effective resistance of each shall be separately computed with reference to the axis of the group in order to determine the factored resistance of the combination.

21.10 Fasteners and Welds in Combination

21.10.1 When approved by the designer, high-strength bolts in slip-resistant connections may be considered as sharing the specified load with welds in the same shear plane in new work. In this case, the factored resistance of the connection, taken as the larger of the individual capacities (high-strength bolts or welds), must also be equal to or greater than the effect of the factored loads.

21.10.2 In making alterations to structures, existing rivets and high-strength bolts may be utilized to carry forces resulting from existing dead loads, and welding may be proportioned to carry all additional loads.

21.11 High-Strength Bolts (in Slip-Resistant Joints) and Rivets, in Combination

In making alterations, rivets and high-strength bolts in slip-resistant joints may be considered as sharing forces due to dead and live loads.

21.12 Connections Requiring High-Strength Bolts or Welds

21.12.1 High-strength bolts or welds shall be used for the following connections:

(a) Connections of beams, girders, and trusses on which the bracing of the structure is dependent, and column splices, in structures over 30 000 mm in height;

(b) Roof truss splices, connections of trusses to columns, column splices, column bracing, knee braces, and crane supports in all structures carrying cranes over 50 kN capacity;

(c) Connections for supports of running machinery, or of other live loads which produce impact or cyclic load;

(d) Any other connections so stipulated on the design drawings.

21.12.2 In all cases except those listed in Clause 21.12.1 connections may be made with A307 bolts.

21.12.3 For the purposes of Clause 21.12, the height of a tier structure is the distance from curb level to the top of the roof beams in flat roofs or curb level to top of roof beams at mean gable height in the case of sloping roofs. Penthouses may be excluded in determining the height of a structure.

21.13 Special Fasteners

Fasteners of special types may be used when approved by the designer.

22. Bolting Details

22.1 High-Strength Bolts

A325M, A490M, A325 and A490 high-strength bolts and their usage shall conform to Clause 23.

22.2 A307 Bolts

Nuts on A307 bolts shall be tightened to an amount corresponding to the full effort of a man using a spud wrench. When so specified, nuts shall be prevented from working loose by the use of lock washers, lock nuts, jam nuts, thread burring, welding, or other methods approved by the designer.

22.3 Effective Bearing Area

The effective bearing area of bolts shall be the nominal diameter multiplied by the length in bearing. For countersunk bolts half the depth of the countersink shall be deducted from the bearing length.

22.4 Long Grips

A307 bolts which carry calculated loads, and with a grip exceeding five diameters, shall have their number increased by 0.6 per cent for each additional 1 mm in the grip.

22.5 Minimum Pitch

The minimum distance between centres of bolt holes preferably shall be not less than 3 bolt diameters and in no case less than 2 2/3 diameters.

22.6 Minimum Edge Distance

The minimum distance from the centre of a bolt hole to any edge shall be that given in Table 14.

22.7 Maximum Edge Distance

The maximum distance from the centre of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the outside connected part with a maximum of 150 mm.

Table 14
Minimum Edge Distance for Bolt Holes

Bolt Diameter		At Sheared Edge	At Rolled, Sawn or Gas Cut Edge†
Inches*	**Millimetres**	**Millimetres**	**Millimetres**
5/8	-	28	22
-	16	28	22
3/4	-	32	25
-	20	34	26
7/8	-	38‡	28
-	22	38	28
-	24	42	30
1	-	44‡	32
-	27	48	34
1 1/8	-	51	38
-	30	52	38
1 1/4	-	57	41
-	36	64	46
Over 1 1/4	Over 36	1 3/4 x Diameter	1 1/4 x Diameter

* ASTM Standards A325 and A490 are written in Imperial Units. Accordingly, bolt diameters are shown in the Imperial System for these bolts only.
† Gas cut edges shall be smooth and free from notches. Edge distance in this column may be decreased 3 mm when hole is at a point where computed stress is not more than 0.3 of the yield stress.
‡ At ends of beam framing angles this distance may be 32 mm.

22.8 **Minimum End Distance**
In the connection of tension members having more than two bolts in a line parallel to the direction of load, the minimum end distance (from centre of end fastener to nearest end of connected part) shall be governed by the edge distance values given in Table 14. In members having either one or two bolts in the line of load, the end distance shall be not less than 1.5 bolt diameters.

22.9 **Slotted Holes**
Maximum and minimum edge distance for bolts in slotted or oversize holes (as permitted in Clause 23.3.2) shall conform to the requirements given in Clauses 22.6, 22.7, and 22.8 assuming that the fastener can be placed at any extremity of the slot or hole.

23. Structural Joints Using ASTM A325M, A490M, A325 or A490 Bolts

23.1 General

23.1.1 Clause 23 deals with the design, assembly, and inspection of structural joints using ASTM A325M, A490M, A325 or A490 bolts, or equivalent fasteners, tightened to a specific minimum tension. A325M, A490M, A325 and A490 bolts are used in holes slightly larger than the nominal bolt size.

23.1.2 Joints required to resist shear between connected parts shall be designated on design drawings and shop details as either bearing-type or slip-resistant.

23.1.3 Slip-resistant shear joints, in which specified load is assumed to be transferred by the slip resistance of the clamped faying surfaces, shall be required where slippage into bearing cannot be tolerated. Such situations may arise in structures subject to fatigue, frequent load reversal, or sensitive to deflection.

23.1.4 In bearing-type shear joints due recognition of the presence or absence of threads in the shear planes of the joint shall be made. Where an outside part adjacent to a nut is less than 10 mm thick, threads shall be considered to be present unless special precautions are taken.

23.1.5 **Applied Tension**
Bolts required to support load by direct tension shall be proportioned so that the tensile load on the bolt area, independent of initial tightening

force, shall not exceed the factored tensile resistance as given in Clause 13.10.2. The applied load shall be taken as the sum of the external load plus any tension caused by prying action due to deformation of the connected parts. If the connection is subject to repeated loading, prying forces must be avoided.

23.1.6 Joints subject to repeated loads shall be proportioned in accordance with Clause 15.

23.2 Bolts, Nuts and Washers*

23.2.1 Except as provided in Clause 23.2.4, bolts, nuts, and washers shall conform to ASTM Standards: A325M, High-Strength Bolts for Structural Steel Joints (Metric); A490M, High-Strength Steel Bolts, Classes 10.9 and 10.9.3 for Structural Steel Joints (Metric); A325, High-Strength Bolts for Structural Steel Joints; A490, Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength.

***Note:** Before specifying metric bolts, the designer should check on their current availability in the quantities required.

23.2.2 The length of bolts shall be such that the point of the bolt will be flush with, or outside the face of, the nut when completely installed.

23.2.3 If required, A325M and A325 bolts, nuts, and washers may be galvanized in accordance with the requirements of ASTM Standards A325M and A325. When installed on a galvanized bolt in a solid steel connection and with three to five threads in the grip, they shall be capable of producing a tensile-type fracture of the bolt and of rotating one full turn from snug before failure.

23.2.4 Other fasteners which meet the chemical composition requirements of ASTM Standards A325M, A490M, A325, or A490 and which meet the mechanical requirements of the same Standard in full-size tests and which have body diameter and bearing areas under the head and nut, or their equivalent, not less than those provided by a bolt and nut of the same nominal dimensions prescribed by Clause 23.2.1 may be used. Such alternative fasteners may differ in other dimensions from the prescribed bolt and nut dimensions. When such fasteners are proposed as an alternative to A325M, A490M, A325 or A490 standard bolts their use shall be subject to the approval of the designer.

23.2.5 If necessary, washers may be clipped on one side to a point not closer than 7/8 of the bolt diameter from the centre of the washer hole.

23.2.6 Design drawings shall indicate the type or types of bolt which may be used. Shop details and erection diagrams shall show the type of bolt to be used.

23.3 Bolted Parts

23.3.1 Bolted parts shall fit together solidly when assembled and shall not be separated by gaskets or any other interposed compressible material.

23.3.2 Holes may be punched, sub-punched or sub-drilled and reamed, or drilled, as permitted by Clause 27.5. The nominal diameter of a hole shall be not more than 2 mm greater than the nominal bolt size, except that, where shown in the design drawings and at other locations approved by the designer, enlarged or slotted holes may be used with high-strength bolts 16 mm in diameter and larger. Joints utilizing enlarged or slotted holes shall be proportioned in accordance with the requirements of Clause 23 and Clauses 13.11 and 13.12 and shall meet the following conditions:

(a) Oversize holes are 4 mm larger than bolts 22 mm and less in diameter, 6 mm larger than bolts 24 mm in diameter, and 8 mm larger than bolts 27 mm and greater in diameter. Oversized holes shall not be used in bearing-type connections but may be used in any or all plies of slip-resistant connections. Hardened washers shall be used under the head and the nut;

(b) Short slotted holes are 2 mm wider than the bolt diameter and have a length which does not exceed the oversize diameter provisions of Item (a) by more than 2 mm. They may be used in any or all plies of slip-resistant or bearing-type connections. The slots may be used without regard to direction of loading in slip-resistant connections but shall be normal to the direction of the load in bearing-type connections. Hardened washers shall be used under the head and the nut;

(c) Long slotted holes are 2 mm wider than the bolt diameter and have a length more than allowed in Item (b) but not more than 2.5 times the bolt diameter and may be used;

(i) In slip-resistant connections without regard to direction of loading. One-third more bolts shall be provided than would be needed to satisfy the requirements of Clause 13.12;

(ii) In bearing-type connections with the long diameter of the slot normal to the direction of loading. No increase in the number of bolts over those necessary in Clause 13.11 is required;

(iii) In only one of the connected parts of either a slip-resistant or bearing-type connection at an individual faying surface;

(iv) Provided that structural plate washers or a continuous bar not less than 8 mm in thickness cover long slots that are in the outer plies of joints. These washers or bars shall have a size sufficient to cover completely the slot after installation.

(d) The above requirement for the nominal diameter of hole may be waived to permit the use of the following bolt diameters and hole combinations in bearing-type of slip-resistant connections:

(i) Either 3/4-inch diameter bolt or an M20 bolt in a 22 mm hole;

(ii) Either 7/8-inch diameter bolt or an M22 bolt in a 24 mm hole;

(iii) Either 1-inch diameter bolt or an M24 bolt in a 27 mm hole.

23.3.3 When assembled, all joint surfaces including those adjacent to bolt heads, nuts, and washers shall be free of scale (tight mill scale excepted), burrs, dirt, and foreign material which would prevent solid seating of the parts.

23.3.4 Faying surfaces within slip-resistant joints shall, for the categories given in Table 4, be as follows:

(a) For categories 1, 4 and 7, free of oil, paint, lacquer, or other coatings;

(b) For category 2, vinyl wash treatment applied in accordance with SSPC Paint 27, Basic Zinc Chromate - Vinyl Butyral Wash Primer, to blast-cleaned surfaces;

(c) For category 3 and 5, zinc-rich paints as defined in SSPC PS Guide 12.00, Guide for selecting

Zinc-Rich Painting Systems, covering zinc-rich paints with organic and inorganic vehicles applied to blast-cleaned surfaces;

(d) For category 6, sprayed metal coatings applied in accordance with CSA Standard G189, Sprayed Metal Coatings for Atmospheric Corrosion Protection;

(e) For category 9, hot-dip galvanizing, provided that faying surfaces are wire brushed or "brush-off" blast-cleaned after galvanizing and prior to assembly.

Faying surfaces within slip-resistant joints also may be coated by other materials and methods provided that these have been sufficiently tested to establish the performance of full-size similarly coated joints to the satisfaction of the designer.

23.4 **Installation**

23.4.1 Each bolt shall be tightened to provide, when all bolts in the joint are tight, at least the minimum bolt tension given in Table 15 for the size and type of bolt used.

23.4.2 Threaded bolts shall be tightened in accordance with Clause 23.5 or 23.6. If necessary, tightening may be done by turning the bolt while holding the nut against rotation.

23.4.3 A325M and A325 bolts may be installed without a hardened washer except as required by Clause 23.3.2(a), (b), or (c) for oversize or slotted holes, or by Clause 23.7.4 (when inspection involves the use of an inspection wrench). A490M and A490 bolts shall be installed with a hardened washer. For A325M, A490M, A325 and A490 bolts, the hardened washer when used shall be under the element (nut or bolt head) turned in tightening. When A490M or A490 bolts are used with steel having a specified minimum yield point of less than 280 MPa a hardened washer shall be placed under the bolt head and under the nut.

23.4.4 Bevelled washers shall be used to compensate for lack of parallelism where, in the case of A325M and A325 bolts, an outer face of bolted parts has more than a 5 per cent slope with respect to a plane normal to the bolt axis. In the case of A490M and A490 bolts, bevelled washers shall be used to compensate for any lack of parallelism due to slope of outer faces.

23.5 "Turn-of-Nut" Tightening

23.5.1 After aligning the holes in a joint, sufficient bolts shall be placed and brought to a "snug-tight" condition to ensure that the parts of the joint are brought into full contact with each other. "Snug-tight" is the tightness attained by a few impacts of an impact wrench or the full effort of a man using a spud wrench.

23.5.2 Following the initial snugging operation, bolts shall be placed in any remaining open holes and brought to "snug-tightness". Re-snugging may be necessary in large joints.

23.5.3 When all bolts are "snug-tight" each bolt in the joint then shall be tightened additionally by the applicable amount of nut rotation given in Table 16, with tightening progressing systematically from the most rigid part of the joint to its free edges. During this operation there shall be no rotation of the part not turned by the wrench unless the bolt and nut are match-marked to enable the amount of relative rotation to be determined.

23.6 Tightening by Use of a Direct Tension Indicator
Tightening by this means is permitted, provided that it can be demonstrated by an accurate direct measurement procedure that the bolt has been tightened in accordance with Table 15.

23.7 Inspection

23.7.1 The inspector shall determine that the requirements of Clauses 23.2, 23.3, 23.4 and 23.5 are met. Installation of bolts shall be observed to ascertain that a proper tightening procedure is employed. The turned element of all bolts shall be visually examined for evidence that they have been tightened. For bearing-type connections with no bolts subject to tensile or combined shear and tensile loads this inspection is all that is required.

23.7.2 Bolts installed by the "turn-of-nut" method may have tensions exceeding those given in Table 15 but this shall not be cause for rejection.

23.7.3 When bolts are installed in accordance with Clause 23.6 the verification that the bolt has been properly tightened is determined by the direct tension indicator.

23.7.4 For bolts in slip-resistant connections and for bolts subject to tensile or combined shear and tension loads, when there is disagreement concerning the results of inspection of bolt tension in the turn-of-nut method, the following arbitration inspection procedure shall be used unless a different procedure has been specified:

(a) The inspector shall use an inspection wrench which shall be a manual or power torque wrench capable of indicating a selected torque value;

(b) Three bolts of the same grade and diameter as those under inspection, and representative of the lengths and condition of those in the structure, shall be placed individually in a calibration device capable of indicating bolt tension. The surface under the part to be turned in tightening each bolt shall be similar to that under the corresponding part in the structure, i.e., there shall be a washer under the part turned if washers are so used in the structure or, if no washer is used, the material abutting the part turned shall be of the same specification as that in the structure;

(c) When the inspection wrench is a manual wrench, each bolt specified in Item (b) shall be tightened in the calibration device by any convenient means first to an initial tension approximately 15 per cent of the required fastener tension and then to the minimum tension specified for its size in Table 15. Tightening beyond the initial condition must not produce greater nut rotation than that permitted in Table 16. The inspection wrench then shall be applied to the tightened bolt and the torque necessary to turn the nut or head 5° in the tightening direction shall be determined. The average torque measured in the tests of three bolts shall be taken as the job inspection torque to be used in the manner specified in Item (e). The job inspection torque shall be established at least once every working day;

(d) When the inspection wrench is a power wrench it shall first be applied to produce an initial tension approximately 15 per cent of the required fastener tension and then adjusted so that it will tighten each bolt specified in Item (b) to a tension at least 5 but not more than 10 per cent greater than the minimum tension specified for its size in Table 15. This setting of wrench shall be taken as the job inspection torque to be used in the manner specified in Item (e). Tightening beyond the initial condition must not produce greater nut rotation than that permitted in Table 16. The job inspection torque shall be established at least once each working day;

(e) Bolts represented by the sample prescribed in Item (b) which have been tightened in the structure shall be inspected by applying, in the tightening direction, the inspecting wrench and its job inspecting torque to 10 per cent of the bolts but not less than two bolts, selected at random in each connection. If no nut or bolt head is turned by this application of the job inspecting torque, the connection shall be accepted as properly tightened. If any nut or bolt head is turned by the application of the job inspecting torque, this torque shall be applied to all bolts in the connection, and all bolts whose nut or head is turned by the job inspecting torque shall be tightened and reinspected. Alternatively, the fabricator or erector at his option may retighten all of the bolts in the connection and then resubmit the connection for the specified inspection.

Table 15
Bolt Tension

Bolt Diameter		Minimum Bolt Tension* (kN)	
Inches	**Millimeters**	**A325M A325**	**A490M A490**
1/2	-	53	67
5/8	-	85	107
-	16	91	114
3/4	-	125	157
-	20	142	178
7/8	-	174	218
-	22	176	220
-	24	205	257
1	-	227	285
-	27	267	334
1 1/8	-	249	356
-	30	326	408
1 1/4	-	316	454
1 3/8	-	378	538
-	36	475	595
1 1/2	-	458	658

* Equal to 70 per cent of specified minimum tensile strength given in the appropriate ASTM specification, soft converted where appropriate and rounded to nearest kilonewton.

Table 16
Nut Rotation* From Snug-Tight Condition

Disposition of Outer Faces of Bolted Parts	Bolt Length ‡	Turn
Both faces normal to bolt axis or one face normal to axis and other face sloped 1:20 (bevel washer not used)†	Up To and Including 4 Diameters	1/3
	Over 4 Diameters and Not Exceeding 8 Diameters or 200 mm	1/2
	Exceeding 8 Diameters or 200 mm	2/3
Both faces sloped 1:20 from normal to bolt axis (bevel washers not used)†	For All Lengths of Bolts	3/4

* Nut rotation is rotation relative to bolt regardless of the element (nut or bolt) being turned. Tolerance on rotation: 30° over or under. For coarse thread heavy hex structural bolts of all sizes and length and heavy hex semi-finished nuts.
† Bevel washers are necessary when A490M or A490 bolts are used.
‡ Bolt length is measured from underside of head to extreme end of point.

24. **Welding**

24.1 **Arc Welding**
Arc welding design and practice shall conform to CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

24.2 **Resistance Welding**
Resistance welding practice and design shall conform to the applicable requirements of CSA Standard W55.2, Resistance Welding Practice. The resistance of resistance welded joints shall be taken as established in CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, and the related welding practice shall be in conformance with welding standards approved by the Canadian Welding Bureau under the same CSA Standard.

24.3 **Fabricator and Erector Qualification**
Fabricators and erectors eligible to assume full responsibility for welded construction covered by this Standard shall be those certified by the Canadian Welding Bureau to the requirements of CSA Standard W47.1, Certification of Companies for Fusion Welding of Steel Structures, for Division 1 or Division 2.1 or CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Building, or both, as applicable. For fusion welded structures, part of the work may be sublet to a Division 3 fabricator or erector; however, full responsibility shall remain with the Division 1 or Division 2.1 fabricator or erector.

25. **Column Bases**

25.1 **Loads**
Suitable provision shall be made to transfer column loads and moments to footings and foundations.

25.2 **Resistance**

25.2.1 **Compressive Resistance of Concrete**
The compressive resistance of concrete shall be determined in accordance with Clause 10.15 of CSA Standard CAN3-A23.3. When compression exists over the entire base plate area the bearing pressure on the concrete may be assumed to be uniform over an area equal to the width of the base plate multiplied by a depth equal to d-2e where e is the eccentricity of the column load.

25.2.2 **Resistance to Pull-Out**

Anchor bolts subject to tensile forces shall be anchored to the foundation unit in such a manner that the required factored tensile force can be developed. Full anchorage is obtained when the factored pull-out resistance of the concrete is equal to or larger than the factored tensile resistance of the bolts. The requirements of CSA Standard CAN3-A23.3 for the transfer of tensile forces from the anchors to the concrete shall be met.

25.2.3 Resistance to Transverse Loads

25.2.3.1 Shear resistance may be developed by friction between the base plate and the foundation unit or by bearing of the anchor bolts or shear lugs against the concrete. When shear acts toward a free edge, the requirements of CSA Standard CAN3-A23.3 shall be met.

25.2.3.2 When loads are transferred by friction, the requirements of CSA Standard CAN3-A23.3 shall be met.

25.2.3.3 When shear is transmitted by bearing of the anchor bolts on the concrete the factored bearing resistance shall be taken as:

$$B_r = 1.40\phi_c nAf'_c$$

where

ϕ_c = 0.60
n = number of anchor bolts in shear
A = bearing area, to be taken as the product of the bolt diameter, d, and an assumed depth of 5d.

25.2.3.4 When shear is transmitted by bearing of shear lugs on the concrete the requirements of CSA Standard CAN3-A23.3 shall be met.

25.2.4 Moment Resistance

Moment resistance shall be taken as the couple formed by the tensile resistance of the anchor bolts determined in accordance with Clause 26.2.1 or 26.2.3 as applicable, and by the concrete compressive resistance determined in accordance with Clause 10.15 of CSA Standard CAN3-A23.3.

25.3 Finishing

Column bases shall be finished in accordance with the following requirements:

(a) Steel-to-steel contact bearing surfaces of rolled steel bearing plates shall be finished in such a manner that the requirements of Clauses 27.8, 27.9.7 and 29.7.3 are satisfied. In general, rolled steel bearing plates 50 mm or less in thickness may be used without planing provided a satisfactory contact bearing is obtained; rolled steel bearing plates over 50 mm but not over 100 mm in thickness may be straightened by pressing or by planing on all bearing surfaces, to obtain a satisfactory contact bearing; rolled steel bearing plates, over 100 mm in thickness, shall be planed on all bearing surfaces except as noted in Clause 25.2(c);

(b) Column bases other than rolled steel bearing plates shall be planed on all bearing surfaces except as noted in Clause 25.2(c);

(c) The bottom surfaces of bearing plates and column bases which rest on masonry or concrete foundations and are grouted to ensure full bearing need not be planed.

26. Anchor Bolts

26.1 General

Anchor bolts shall be designed to resist the effect of factored uplift forces, bending moments and shears determined in accordance with Clause 7.2. The anchorage of the anchor bolts in the foundation unit shall be such that the required load capacity can be developed. Forces present during construction as well as those present in the finished structure shall be resisted.

26.2 Bolt Resistance

26.2.1 Tension

The factored tensile resistance of an anchor bolt shall be taken as:

$$T_r = \phi_b A_n F_u$$

where A_n = the tensile stress area of the bolt

$$= \frac{\pi}{4} (D - 0.97p)^2$$

ϕ_b = 0.67
p = the pitch of thread in mm

26.2.2 Shear

The factored shear resistance of the anchor bolts shall be taken as:

$$V_r = 0.60\phi_b n A_b F_u$$

but not greater than the lateral bearing resistance given in Clause 25.2.3.3.

When the bolt threads are intercepted by the shear plane the factored shear resistance shall be taken as 70 percent of V_r.

26.2.3 **Shear and Tension**

An anchor bolt required to develop resistance to both tension and shear shall be proportioned so that

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \leq 1$$

where V_f is the portion of the total shear per bolt transmitted by bearing of the anchor bolts on the concrete. See Clause 25.2.3.3.

26.2.4 **Tension and Bending**

An anchor bolt required to develop resistance to both tension and bending shall be proportioned to meet the requirements of Clause 13.8(a). The tensile and moment resistances T_r and M_r, shall be based on the properties of the cross section at the critical section, M_r shall be taken as $\phi_b S F_y$.

27. Fabrication

27.1 **General**

Unless otherwise specified, workmanship shall be in accordance with prevailing practice and the provisions of Clause 27 shall apply to both shop and field fabrication.

27.2 **Straightness of Material**

Prior to layout of fabrication, rolled material shall be straight within established rolling mill tolerances. Straightening or flattening shall be done by means that will not injure the material and protective coatings, if present. Sharp kinks and bends shall be cause for rejection.

27.3 **Gas Cutting**

Gas cutting shall be done by machine where practicable. Gas cut edges shall conform to CSA Standard W59, Welded Steel Construction (Metal-Arc Welding). Re-entrant corners shall be free from notches and shall have the largest practical radii, with a minimum radius of 14 mm.

27.4 **Sheared or Gas Cut Edge Finish**

27.4.1 Planing or finishing of sheared or gas cut edges of plates or shapes shall not be required unless specifically noted on the drawings or included in a stipulated edge preparation for welding.

27.4.2 The use of sheared edges in the tension area shall be avoided in locations subject to plastic hinge rotation at factored loading. If used, such edges shall be finished smooth by grinding, chipping, or planing.

27.4.3 Burrs shall be removed as required in Clause 23.3.3, and when required for proper fit-up for welding, and when burr creates a hazard during or after construction.

27.4.4 The requirements of Clause 27.4.2 shall be noted on design and shop drawings when applicable.

27.5 Holes for Bolts or Other Mechanical Fasteners

27.5.1 Unless otherwise shown on design drawings or as specified in Clause 23.3.2, holes shall be made 2 mm larger than the nominal diameter of the fastener. Holes may be punched when the thickness of material is not greater than the nominal fastener diameter plus 4 mm. For greater thicknesses holes shall be drilled from the solid or either sub-punched or sub-drilled and reamed. The die for all sub-punched holes or the drill for all sub-drilled holes shall be at least 4 mm smaller than the required diameter of the finished hole. Holes in CSA Standard G40.21-M (Type 700Q) or ASTM Standard A514 steels over 13 mm in thickness shall be drilled.

27.5.2 In locations subject to plastic hinge rotation at factored loading, fastener holes in the tension area shall be sub-punched and reamed or drilled full size.

27.5.3 The requirements of Clause 27.5.2 shall be noted on design and shop drawings where applicable.

27.6 Bolted Construction

27.6.1 Drifting necessary during assembly to align holes shall not distort the metal nor enlarge the holes. Holes in adjacent parts shall match sufficiently well to permit easy entry of bolts. If necessary, holes, except oversize or slotted holes, may be enlarged to admit bolts by a moderate amount of reaming; however, gross mismatch of holes shall be cause for rejection.

27.6.2 Assembly of high-strength bolted joints shall be in accordance with Clause 23.

27.7 **Welded Construction**

Workmanship and technique in arc-welded fabrication shall conform to those prescribed by CSA Standard W59, Welded Steel Construction (Metal-Arc Welding). The welding practice in resistance welded fabrication shall conform to that required by CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, and approved by the Canadian Welding Bureau.

27.8 **Finishing of Bearing Surfaces**

Compression joints which depend on contact bearing shall have the bearing surfaces prepared to a common plane by milling, sawing, or other suitable means. Surface roughness shall have a roughness height rating not exceeding 500 (12.5 μm) as defined in CSA Standard B95, Surface Texture (Roughness, Waviness, and Lay), unless otherwise specified.

27.9 **Tolerances**

27.9.1 Structural members consisting primarily of a single rolled shape shall be straight within the tolerances allowed by CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel, except as specified in Clause 27.9.4.

27.9.2 Built-up bolted structural members shall be straight within the tolerances allowed for rolled wide-flange shapes by CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel, except as specified in Clause 27.9.4.

27.9.3 Dimensional tolerances of welded structural members shall be those prescribed by CSA Standard W59, Welded Steel Construction (Metal-Arc Welding), unless otherwise specified.

27.9.4 Fabricated compression members shall not have a deviation from straightness more than one-thousandth of the axial length between points which are to be laterally supported.

27.9.5 Beams with bow within straightness tolerances shall be fabricated so that after erection the bow due to rolling or fabrication shall be upward.

27.9.6 All completed members shall be free from twists, bends, and open joints. Sharp kinks or bends shall be cause for rejection.

27.9.7 Compression joints which depend upon contact bearing, when assembled during fabrication, shall have at least 75 per cent of the entire contact area in full bearing and the separation of any remaining portion shall not exceed 0.25 mm except adjacent to toes of flanges where a localized separation not exceeding 0.60 mm is permissible.

27.9.8 A variation of 1 mm is permissible in the overall length of members with both ends finished for contact bearing.

27.9.9 Members without ends finished for contact bearing, which are to be framed to other steel parts of the structure, may have a variation from the detailed length not greater than 2 mm for members 10 000 mm or less in length and not greater than 4 mm for members over 10 000 mm in length.

28. <u>Cleaning, Surface Preparation and Priming</u>

28.1 General Requirements

28.1.1 All steelwork, except as exempted by Clauses 28.1.2, 28.1.3, and 28.2 or unless otherwise noted on design drawings or in the job specifications, shall be given one coat of primer or one-coat paint (see Clause 28.5) applied in the shop. The primer or one-coat paint shall be applied thoroughly and evenly to dry clean surfaces by suitable means.

28.1.2 Steelwork to be subsequently concealed by interior building finish need not be given a coat of primer unless otherwise specified (see Clause 6.4.2).

28.1.3 Steelwork to be encased in concrete need not be given a coat of primer. Steelwork designed to act compositely with reinforced concrete and depending on natural bond for interconnection shall not be given a coat of primer.

28.1.4 Steelwork to be shop-primed shall be cleaned of all loose mill scale, loose rust, weld slag, and flux deposit, dirt, and other foreign matter and excessive weld spatter prior to application of the primer. Oil and grease shall be removed by solvent. The fabricator shall be free to use any satisfactory method to clean the steel and prepare the surface for painting unless a specific method of surface preparation is called for.

28.1.5 Primer shall be dry before loading primed steelwork for shipment.

28.1.6 Steelwork not to be shop-primed after fabrication shall be cleaned of oil and grease by solvent cleaners and shall be cleaned of dirt and other foreign matter.

28.2 Requirements for Special Surfaces

28.2.1 Surfaces inaccessible after assembly shall be cleaned, or cleaned and primed, as required by Clause 28.1, prior to assembly. Inside surfaces of enclosed spaces entirely sealed off from any external source of oxygen need not be primed.

28.2.2 In compression members, surfaces finished to bear and assembled during fabrication shall be cleaned before assembly but shall not be primed unless otherwise specified.

28.2.3 Surfaces finished to bear and not assembled during fabrication shall be protected by a corrosion inhibiting coating. The coating shall be of a type that can be readily removed prior to assembly or shall be of a type that makes such removal unnecessary.

28.2.4 Faying surfaces of high-strength bolted slip-resistant joints shall not be primed or otherwise coated except as permitted by Clause 23.

28.2.5 Joints to be field welded and surfaces to which shear connections are to be welded shall be kept free of primer and any other coating which could be detrimental to achieving a sound weldment, except that sheet steel decks may be welded to clean primed steelwork.

28.3 Surface Preparation

Unless otherwise specified, or approved, surface preparation shall be in conformance with one of the following applicable specifications of the Steel Structures Painting Council:

SP 2
Hand Tool Cleaning;

SP 3,
Power Tool Cleaning;

SP 4,
Flame Cleaning of New Steel;

SP 5,

White Metal Blast Cleaning;

SP 6,
Commercial Blast Cleaning;

SP 7,
Brush-Off Blast Cleaning;

SP 10,
Near-White Blast Cleaning.

28.4 **Primer**
Unless otherwise specified, or approved, shop primer shall conform to one of the following standards of the Canadian General Standards Board:

1-GP-14e,
Primer: Red Lead in Oil;

1-GP-40d,
Primer: Structural Steel, Oil Alkyd Type;

1-GP-81e,
Primer, Alkyd, Air Drying and Baking, for Vehicles and Equipment;

1-GP-140c,
Primer: Red Lead, Iron Oxide, Oil Alkyd Type;

1-GP-166a,
Primer: Basic Lead Silico-Chromate, Oil Alkyd Type;

CISC/CPMA 2-75,
A Quick-Drying Primer For Use On Structural Steel.

28.5 **One-Coat Paint**
Unless otherwise specified, or approved, one-coat paint intended to withstand exposure to essentially non-corrosive atmosphere for a period of time not exceeding 6 months shall conform to CISC/CPMA Standard 1-73a, A Quick-Drying One-Coat Paint For Use On Structural Steel.

29. **Erection**

29.1 **General**
The steel framework shall be erected true and plumb within the specified tolerances. Temporary bracing shall be employed wherever necessary to withstand all loads to which the structure may be subject during erection and subsequent construction, including loads due to wind, equipment and operation of same. Temporary bracing shall be left in place undisturbed as long as required for the safety and

integrity of the structure (see also Clause 26). The erector shall ensure during erection that an adequate margin of safety exists in the uncompleted structure and members using the factored member resistances computed in accordance with Clause 13 (see also Clause 20.1.3).

29.2 **Marking of Members**
Erection marks or other suitable means shall be used to identify components. Erection marks which are injurious to the material or to finished surfaces exposed to view shall be avoided.

29.3 **Handling**
Adequate care shall be taken to avoid damage during handling, especially for long slender members. Injury to protective coatings shall be avoided.

29.4 **Temporary Loads**
Wherever piles of material, erection equipment, or other loads are carried during erection, suitable provision shall be made to ensure that the loads can be safely sustained during their duration and without permanent deformation or other damage to any member of the steel frame and other building components supported thereby.

29.5 **Adequacy of Temporary Connections**
As erection progresses, the work shall be securely bolted or welded to take care of all dead load, wind, and erection loads, and to assist in providing structural integrity.

29.6 **Alignment**
No permanent welding or bolting shall be done until as much of the structure as will be stiffened thereby has been suitably aligned.

29.7 **Surface Preparation for Field Welding**
The portions of surfaces that are to receive welds shall be thoroughly cleaned of all foreign matter, including paint film.

29.8 **Field Painting**
Unless otherwise specified, the cleaning of steelwork in preparation for field painting, touch-up of shop primer, spot-painting of field fasteners, and general field painting, shall not be considered to be a part of the erection work.

29.9 **Erection Tolerances**

29.9.1 Unless otherwise specified, members of the steel framework shall be considered plumb, level, and aligned if the misalignment does not exceed the following tolerances:

(a) Exterior columns of multi-storey buildings - 1 to 1000; but not more than 25 mm towards nor 50 mm away from the building line in the first 20 storeys plus 2 mm for each additional storey up to a maximum of 50 mm towards or 75 mm away from the building line over the full height of the building;

(b) Columns adjacent to elevator shafts - 1 to 1000; but not more than 25 mm in the first 20 storeys plus 1 mm for each additional storey up to a maximum of 50 mm over the full height of the elevator shaft;

(c) Spandrel beams - 1 to 1000;

(d) All other pieces - 1 to 500.

29.9.2 Shelf angles, sash angles, and lintels specified to be provided with adjustable connections shall be considered within tolerances when each piece is level within a tolerance of 1 to 1000, when adjoining ends of these members are aligned vertically within 2 mm and when the locations of these members vertically and horizontally is within 10 mm of the location established by the dimensions on the drawings.

29.9.3 Column splices and other compression joints which depend upon contact bearing shall, after alignment, have at least 65 per cent of the entire contact area in full bearing and the separations of any remaining portions shall not exceed 0.5 mm except locally at toes of flanges, where a separation of 0.75 mm is permissible; otherwise corrective measures shall be taken.

29.9.4 The fit-up of joints to be field welded shall be within the tolerances shown on the field assembly drawings before welding is begun.

30. Inspection

30.1 General

Material and workmanship at all times shall be subject to inspection by qualified inspectors representing and responsible to the designer. The inspection shall cover shop work and field erection work to ensure compliance with this Standard.

30.2 Co-operation

All inspection insofar as possible shall be made in the fabricator's shop and the fabricator shall co-operate with the inspector, permitting access for inspection to all places where work is being done. The inspector shall co-operate in avoiding undue delay in the fabrication or erection of the steelwork.

30.3 Rejection

Material or workmanship not conforming to the provisions of this Standard may be rejected at any time during the progress of work when non-conformance to these provisions is established.

30.4 Inspection of High-Strength Bolted Joints

The inspection of high-strength bolted joints shall be performed in accordance with the procedures prescribed in Clause 24.

30.5 Inspection of Welding

The inspection of welding shall be in accordance with the applicable clause in CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

30.6 Identification of Steel by Marking

In the fabricator's plant, steel used for main components shall at all times be marked to identify its specification (and grade, if applicable). This shall be done by suitable markings or by recognized colour coding except that cut pieces identified by piece mark and contract number need not continue to carry specification identification marking when it has been satisfactorily established that such cut pieces conform to the required material specifications.

Appendix A

Standard Practice for Structural Steel for Buildings

Note: This Appendix is not a mandatory part of this Standard.

A1. Matters concerning standard practice not covered by the Standard but pertinent to the fabrication and erection of structural steel, such as a definition of structural steel items, the computation of weights, etc., should be clearly specified in the plans and specifications issued to the bidders, or in accordance with a specification like the "Code of Standard Practice for Structural Steel" published by the Canadian Institute of Steel Construction.

Appendix B

Effective Lengths of Compression Members in Frames

Note: This Appendix is not a mandatory part of this Standard.

B1. The slenderness ratio of a compression member is defined as the ratio of the effective length to the applicable radius of gyration. The effective length KL may be thought of as the actual unbraced length L multiplied by a factor K such that the product KL is equal to the length of a pin-ended compression member of equal capacity to the actual member. The effective length factor K of a column of finite unbraced length is therefore dependent upon the conditions of restraint afforded to the column at its braced locations and theoretically may vary from 0.5 to infinity. In practical building applications, K would be somewhat greater than 0.5 in the most favourable situation and in all probability would not exceed 5 in the most unfavourable situation.

B2. A variation in K between 0.65 and 2.0 would apply to the majority of cases likely to be encountered in actual structures.

B3. When proportioning columns on the basis of effective lengths the designer is presented with two basic situations which have a pronounced effect upon the strength of axially loaded columns.

(a) For structures in which the sway effects have been included in the analysis to determine the design moments and forces, the effective length factor is determined from the degree of rotational restraint afforded at the ends of the unbraced length and K will be equal to or less than 1.0. In Appendix C this case is identified as the side-sway prevented case;

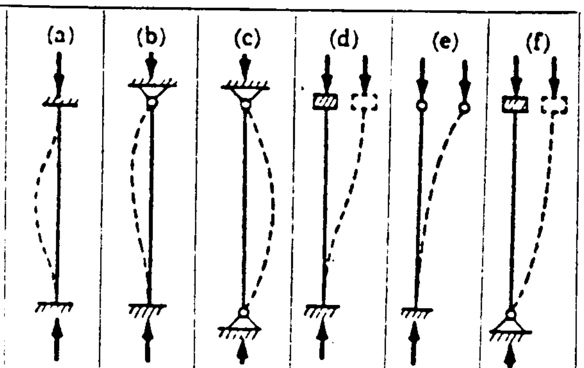
(b) For structures in which the sway effects have not been included in the analysis to determine the design moments and forces, the effective length factor is determined from the degree of rotational and translational restraint afforded at the ends of the unbraced length and K will be equal to or greater than 1.0. In Appendix C this case is identified as the side-sway permitted case.

B4. Figure B1 illustrates six idealized cases in which joint rotation and translation are either fully realized or non-existent.

B5. For a frame with columns pinned at their bases, Figure B2 shows diagrammatically the difference in effective column length when the sway effect is and is not included in the analysis to determine the design moments and forces. In the former case, the effect of the loads acting at the displacement Δ_b has been included in the analysis and the effective length, K_aL is based on the sway prevented condition ($K_aL < L$) which accounts only for the effect of the displacement Δ_a on column stability. In the latter case, the effective length K_bL is based on the sway permitted condition ($K_bL > L$) in an attempt to include the influence of the displacement Δ_b on column stability.

B6. The use of the sway permitted case is approximate only as the moments and forces due to the sway effects are not taken into account in the design of the girders. In frames that depend on means other than frame action to achieve stability, such as bracing, the bracing shall be designed to resist the combined effects of the superimposed loads plus the loads due to the sway effects.

B7. In Figure B2 the column bases are shown to be pinned and G_L would theoretically be infinity. In practical situations, however, the restraining effect of the normal flat-ended column base detail exerts a beneficial influence on the true effective length of the column, even where the footing is designed only for vertical load. Thus in most cases G_L can be taken as 10 (or less where justified) in the computation of K.

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.0	2.0

End condition code 

Rotation fixed	Translation fixed
Rotation free	Translation fixed
Rotation fixed	Translation free
Rotation free	Translation free

Figure B1

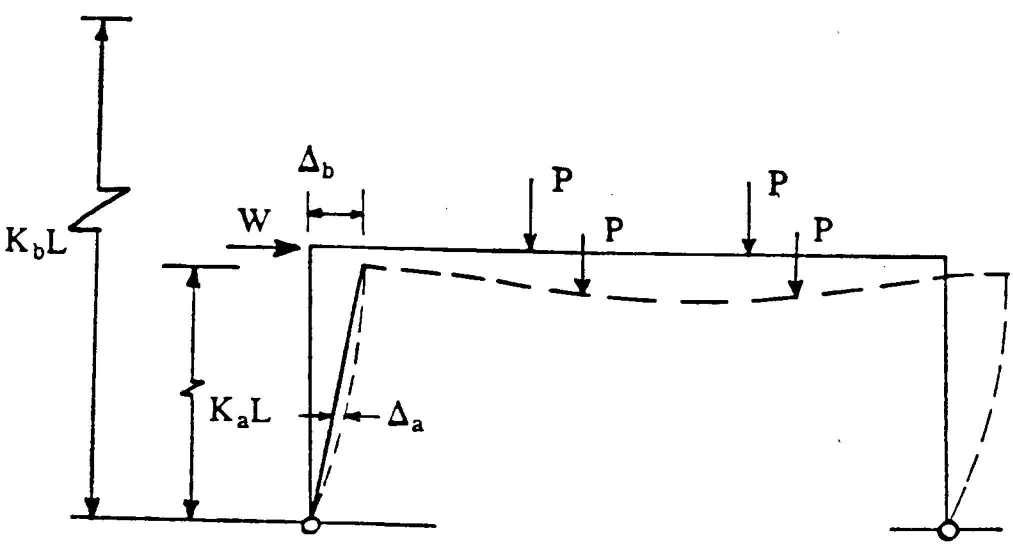


Figure B2

Appendix C

Criteria for Estimating Effective Column Lengths in Continuous Frames

Note: This Appendix is not a mandatory part of this Standard.

C1. Two cases influencing the design of columns in continuous frames are considered:

(a) Sway effects included in the analysis - side-sway prevented;

(b) Sway effects not included in the analysis - side-sway permitted.

C2. Figure C1 is a nomograph applicable to cases in which the equivalent I/L of adjacent girders which are rigidly attached to the columns are known, and is based on the assumption that all columns in the portion of the framework considered reach their individual critical loads simultaneously.

In the usual building frame not all columns would be loaded so as to simultaneously reach their buckling loads, and thus some conservatism is introduced in the interest of simplification.

C3. The equations upon which these nomographs are based are:

(a) Side-sway prevented:

$$\frac{G_U G_L}{4}(\pi/K)^2 + \left(\frac{G_U + G_L}{2}\right)\left(1 - \frac{\pi/K}{\tan \pi/K}\right) + 2\,\frac{\tan \pi/2K}{\pi/K} = 1$$

(b) Side-sway permitted:

$$\frac{G_U G_L (\pi/K)^2 - 36}{6(G_U - G_L)} = \frac{\pi/K}{\tan \pi/K}$$

C4. Subscripts U and L refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\Sigma I_c/L_c}{\Sigma I_g/L_g}$$

in which Σ indicates a summation for all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered, I_c is the moment of inertia and L_c the unsupported length of a column section, and I_g is the moment of inertia and L_g the unsupported length of a girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered.

C5. For column ends supported by, but not rigidly connected to, a footing or foundation, "G" may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, "G" may be taken as 1.0. Smaller values may be used if justified by analysis.

C6. Refinements in girder I_g/L_g may be made when conditions at the far end of any particular girder are known definitely or when a conservative estimate can be made. For the case with no side-sway, multiply girder stiffnesses by the following factors:

1.5 for far end of girder hinged;
2.0 for far end of girder fixed against rotation; (i.e., rigidly attached to a support which is itself relatively rigid).

C7. For the case with side-sway permitted, multiply girder stiffnesses by 0.5 for far end of girder hinged.

C8. Having determined G_U and G_L for a column section, the effective length factor K is determined by constructing a straight line between the appropriate points on the scales for G_U and G_L.

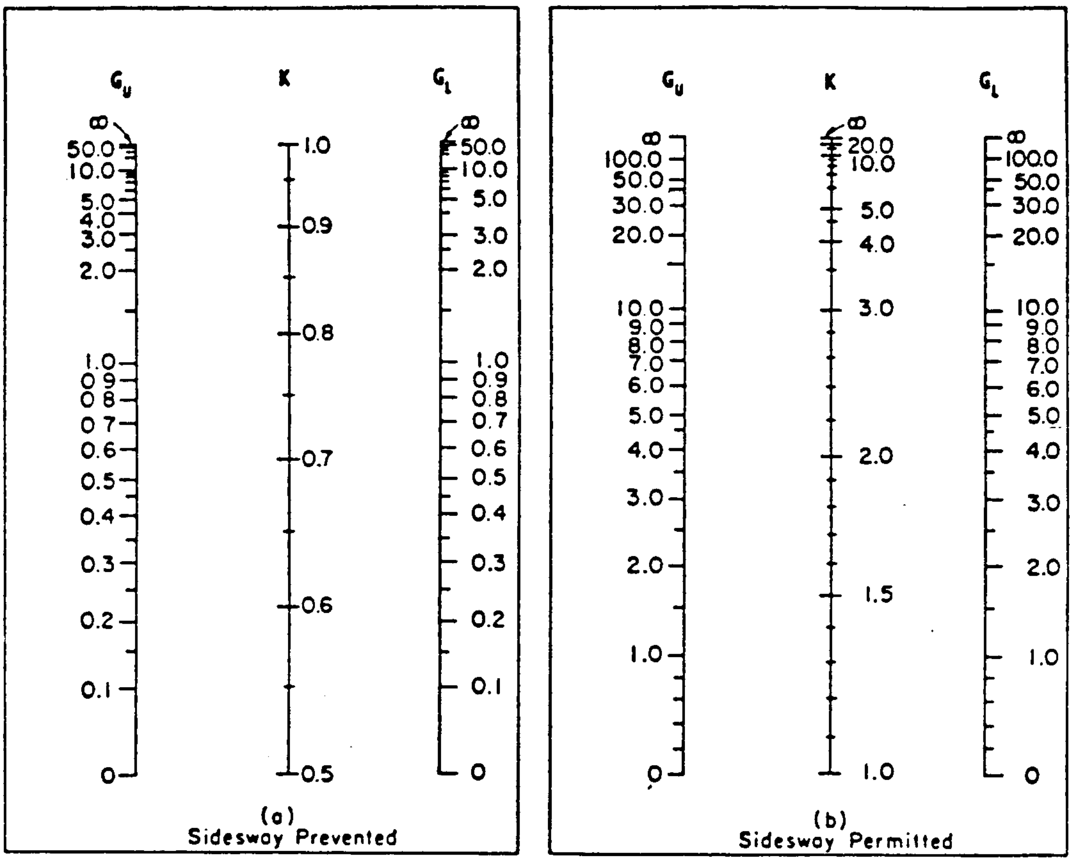


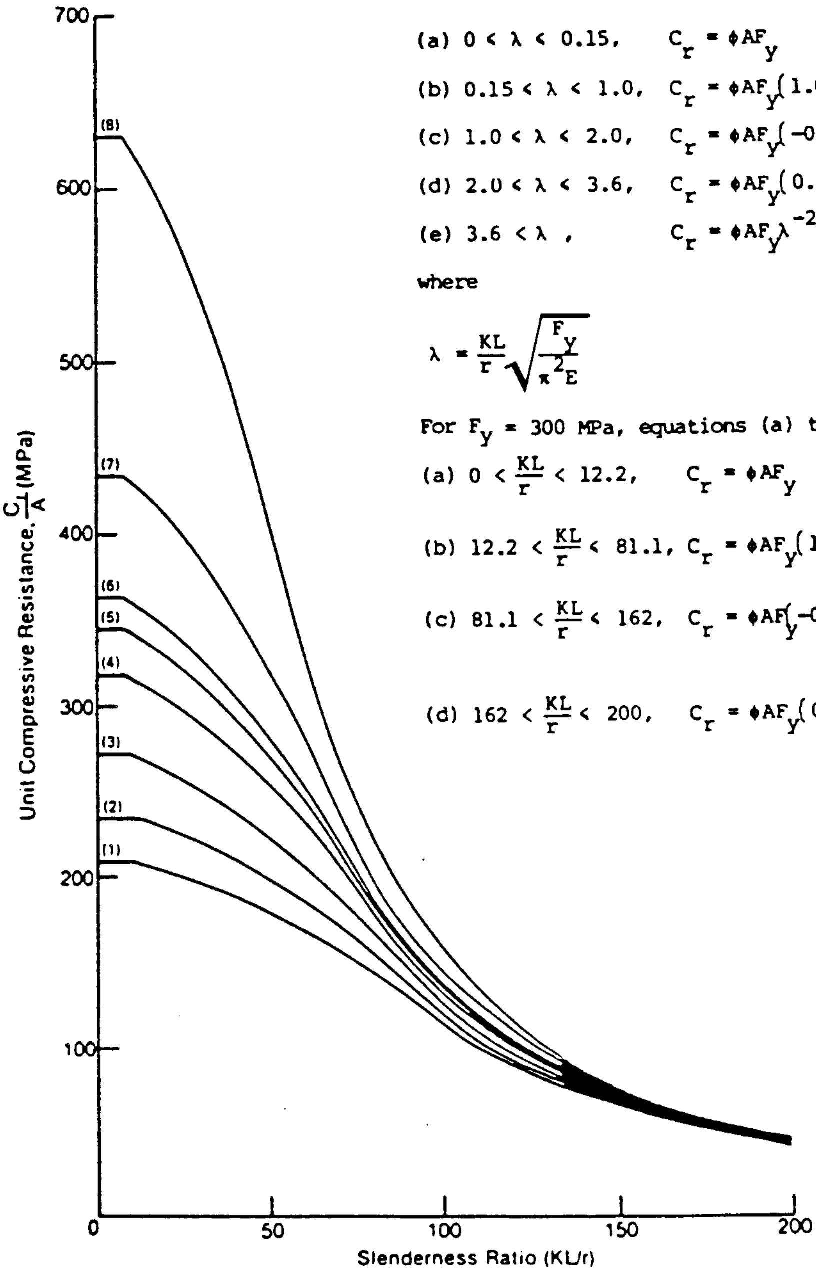
Figure C1

Alignment Chart for Effective Length of Columns in Continuous Frames

Appendix D

Graph Showing Unit Compressive Resistance Versus Slenderness Ratio

Note: This Appendix is not a mandatory part of this Standard.



(a) $0 < \lambda < 0.15, \quad C_r = \phi A F_y$

(b) $0.15 < \lambda < 1.0, \quad C_r = \phi A F_y(1.035 - 0.202\lambda - 0.222\lambda^2)$

(c) $1.0 < \lambda < 2.0, \quad C_r = \phi A F_y(-0.111 + 0.636\lambda^{-1} + 0.087\lambda^{-2})$

(d) $2.0 < \lambda < 3.6, \quad C_r = \phi A F_y(0.009 + 0.877\lambda^{-2})$

(e) $3.6 < \lambda, \quad C_r = \phi A F_y \lambda^{-2}$

where

$$\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}$$

For $F_y = 300$ MPa, equations (a) to (d) become

(a) $0 < \frac{KL}{r} < 12.2, \quad C_r = \phi A F_y$

(b) $12.2 < \frac{KL}{r} < 81.1, \quad C_r = \phi A F_y\left(1.035 - 0.00249\frac{KL}{r} - 0.0000337\left(\frac{KL}{r}\right)^2\right)$

(c) $81.1 < \frac{KL}{r} < 162, \quad C_r = \phi A F_y\left(-0.222 + \frac{51.6}{(KL/r)} + \frac{572}{(KL/r)^2}\right)$

(d) $162 < \frac{KL}{r} < 200, \quad C_r = \phi A F_y\left(0.009 + \frac{5770}{(KL/r)^2}\right)$

Plot	F_y (MPa)
(1)	230
(2)	260
(3)	300
(4)	350
(5)	380
(6)	400
(7)	480
(8)	700

Figure D1

Note: For the curves plotted, equation (e) applies only to F_y = 700 Mpa with KL/r > 191.2.

Appendix E

Margins of Safety

Note: This Appendix is not a mandatory part of this Standard

E1. An advantage of limit states design is that the probability of failure for different loading conditions is made more consistent, by the use of distinct load factors for the different loads to which the structure is subject, than is the case in working stress design where a single factor of safety is used. Furthermore different resistance factors can, in a parallel manner, be applied to determine member resistances with a uniform reliability. The combination of the load factor and the inverse of the resistance factor gives a number comparable to the traditional factor of safety. In this Standard a resistance factor of 0.90 is generally used.

E2. For live loads the load factor of 1.50 multiplied by the inverse of the resistance factor 1/0.90 equals 1.67, comparable to the working stress design standard. By using a load factor of 1.25 for dead load, probabilistic studies indicate that consistent probabilities of failure are determined over all ranges of dead to live load ratios. The same probabilistic studies also show that load combination factors of 0.70 and 0.60, (depending on the number of loads taken in combination) applied only to live, wind or earthquake and temperature, and a factor of 0.85 applied to dead load when it is counteractive to live loads, also result in a consistent probability of failure.

E3. The resistance factor, see Clause 3.1, generally allows for underrun in the member or connection resistance as compared to that predicted. The underrun may arise from variability in material properties, dimensions and workmanship as well as from simplifications in the mathematical derivation of the resistance equation.

For simplicity in some cases in this Standard uncertainty in the formulation of the theoretical member resistance has been incorporated directly in the expression for member resistance rather than using a lower value for the resistance factor. This is the case for the column curve where the curve predicting the ultimate strengths as a function of slenderness ratio has been derived statistically taking into account residual stresses and initial out-of-straightness.

E4. For bolts a resistance factor of 0.67 is used to ensure that connector failures will not occur before general failure of the member as a whole. For long bolted joints and for the case where shear planes intersect the threads, reduction factors are applied to the resistance formulations. As for bolts, a resistance factor of 0.67 is desirable for welds.

E5. The expressions for resistance for welds are based on CSA Standard W59, Welded Steel Construction (Metal-Arc Welding), and therefore maintain the margins of safety of that Standard.

Appendix F

Guide for Floor Vibrations

Note: This Appendix is not a mandatory part of this Standard.

F1. Recent developments of floors of lighter construction, longer spans and less inherent damping have sometimes resulted in problems of objectionable floor vibrations during normal human activity. Fatigue or overloading of floor structures due to vibration is not covered in the Appendix. Some guidance on this is given in Reference 1 for assembly occupancies.

F2. Two types of vibration problems arise in floor construction. Continuous vibrations arise due to the periodic forces of machinery, vehicles or certain human activities such as dancing. These vibrations can be considerably amplified when the periodic forces are synchronized with a floor frequency - a condition called resonance. Transient vibrations, which decay as shown in Fig. F1, arise due to footsteps or other impact.

F3. The most important floor characteristics affecting vibration problems are the natural frequency in hertz (cycles per second) - usually that corresponding to the lowest mode of vibration - and damping. The relation between damping, expressed in per cent of critical damping[2], and decay of free vibration is shown in Figure F2. Other characteristics affecting transient vibration problems are mass, especially for heavy long span floors, and stiffness under point load, especially for light short span floors.

F4. **Thresholds of Annoyance**

F4.1 Generally people do not like floors to vibrate. For continuous sinusoidal vibration lasting more than about ten cycles an average threshold of definite perception is shown in Figure F3 in terms of peak acceleration; the threshold levels for different people range from about one-half to twice the level shown. In the frequency range 2-8 Hz, where people are most sensitive to vibration, the threshold corresponds to 0.5 per cent g approximately, where g is the acceleration due to gravity. The threshold of definite perception shown in Figure F3 can be used to approximate a design threshold of annoyance for residential, school and office occupancies; the design level will be lower for sensitive occupancies (e.g. operating rooms, special laboratories) and greater for industrial occupancies.

F4.2 For transient vibrations, the design threshold in terms of initial peak acceleration of a decaying vibration, as shown in Figure F1, increases with an increase in damping. This is because people find continuous vibration much more annoying than vibration which quickly dies out.

F4.3 Design thresholds equivalent to that for continuous vibration are shown in Figure F3 for transient vibrations due to footsteps (walking vibrations) for different levels of damping.[3]

F5. Continuous Vibrations - Resonance

F5.1 Continuous vibrations caused by machines can be reduced by special design provisions[2,4] such as vibration isolation. Care should be taken at the planning stage to locate such machinery away from sensitive occupancies such as offices.

F5.2 Floor vibrations can also rise from heavy street traffic on bumpy pavement over soft subgrade. The annoyance increases considerably when repetitive vehicles such as buses create ground vibrations which synchronize with the floor frequency.

F5.3 Continuous vibrations caused by human activities may be a problem for light residential floors, or for long span floors used for special purposes such as dancing, concerts or gymnastics. People alone or in union can create periodic forces in the frequency range 1-4 Hz approximately, and therefore for such occupancies, natural frequencies less than 5 Hz should be avoided. To avoid very noticeable vibration for very repetitive activities such as dancing, it is recommended that the frequency of such floors be 8 Hz or more. See Reference 1 for more specific guidance on vibrations due to rhythmic human activities.

F6. Transient Vibrations

F6.1 Objectionable vibration due to footstep impact can occur in floor systems with light damping in residential, school, office and similar occupancies. Because this is the most common source of annoyance, the remainder of this guide will be concerned with this problem. Types of construction which may give transient vibration problems include open web steel joists or steel beams with concrete deck and light wood deck floors using steel joists.

F7. **Performance Test for Floor Vibration**

F7.1 The vibration acceptability of a floor system to human activity can be evaluated by a performance test. Partitions, rug and furnishings, finishes, etc., contribute to reduce vibration annoyance and should therefore be considered in setting up the test floor. A measuring device, which filters out frequencies greater than approximately 1.5 times the fundamental frequency, should be located near mid-span. A person who will give a subjective evaluation of the floor should also be sitting close to the measuring device.

F7.2 One test is for a person of average weight with softsoled shoes to rise up on his toes and drop on his heels near the location of measurement. Fundamental frequency, damping from the decay record (see Figure F2) and peak acceleration are obtained form the measurement and the peak acceleration is plotted on Figure F3 to see how it compares with the threshold of annoyance. Another test is to check floor comfort when different persons walk down the floor; the average peak acceleration can then be compared with the annoyance threshold for steady motion given in Figure F3.

F8. **Long Span Steel Floors With Concrete Deck**

F8.1 Transient vibrations may be a problem for open web steel joists or steel beams with concrete deck, composite or non-composite, generally of spans 7000-20 000 mm and frequencies in the range 4-15 Hz. For such floors, partitions, if properly located, provide more than enough damping to avoid excessive vibrations. On the other hand transient vibrations may be serious for bare floors with very low inherent damping, as is the case for fully composite construction. Figure F3 shows that the threshold of annoyance is roughly 10 times greater for 12 per cent damping than for 3 per cent damping.

F8.2 To assess vibration acceptability requires a knowledge of frequency, damping and peak acceleration from heel impact. If design by performance testing is not feasible, these parameters should be estimated by calculation as follows:

(a) **Frequency** can be estimated by assuming full composite action, even for non-composite construction. For a simply-supported one-way system, the frequency f_1 is given by:

$$f_1 = 156 \sqrt{\frac{EI_T}{wL^4}} \quad (1)$$

where E is the modulus of elasticity of steel (200 000 MPa), I_T the moment of inertia (mm^4) of the transformed T section (concrete transformed to steel) assuming a concrete flange of width equal to the spacing of steel joists or beams, L the span in millimetres and w the dead load of the T-section in N/mm of span. Often one-way systems are supported on steel girders, and this can reduce the frequency calculated for a one-way system.

In this case the frequency can be approximated by:

$$\frac{1}{f^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2} \quad (2)$$

where f_2 is the frequency of floor supported on steel girder perpendicular to joists. A continuous beam of equal spans on flexible supports should be treated as simply-supported since adjacent spans vibrate in opposite directions. For other conditions of span and restraint the dynamically equivalent simply-supported span is less than the full span and can be estimated from the fundamental mode shape.

(b) **Damping** is generally more difficult to estimate than frequency. A bare steel and concrete deck floor has a damping of approximately 3-4 per cent critical for non-composite construction, and about 2 per cent for fully composite construction. The addition of components such as floor finishing, rug and furnishings, ceiling, fireproofing and ducts increases the damping by about 3 per cent or more. Partitions, either above or below the floor, provide the most effective damping especially when they are located in both directions. Even light partitions which do not extend to the ceiling provide considerable damping. Partitions along with supports, or parallel to the floor joists and further apart than approximately 6000 mm, however, may not be effective because the nodal lines of vibration form under the partitions. Human beings also provide damping but this is less effective for heavy long span floors than for lighter short span floors. The following values are suggested for design calculation:

	Damping in Per Cent Critical
Bare floor	3
Finished floor - ceiling, ducts, flooring, furniture	6
Finished floor with partitions	12

(c) **Peak acceleration from heel drop** for floors greater than 7000 mm span and frequencies less than about 10 Hz, can be estimated by assuming an impulse of 70 N•s suddenly applied to a simple spring and mass system, whose mass gives the same response as that of the floor system represented as a simply-supported beam vibrating in the fundamental mode. The peak acceleration, a_o in per cent g, can then be approximated by[3]:

$$a_o = \frac{(0.9)\ 2\pi f \times \text{impulse}}{\text{equivalent mass}} = \frac{60f}{wBL} \qquad (3)$$

where f is the frequency in hertz, w the weight of the floor plus contents in kPa, L the span and B the width of the equivalent beam, both in metres.

For steel joist or beam and concrete deck systems on stiff supports, L is the joist span and B can be approximated as 40 t_c, where t_c is the thickness of concrete deck determined from the average weight of concrete, inlcuding ribs. For joists or beams and concrete deck supported on flexible girders, where the girder frequency is much less than joist frequency and therefore girder vibration predominates, L is the girder span and B can be approximated as the width of floor supported by the girder. For cases where both frequencies are similar, Eqn. (2) can be used to determined frequency and BL can be estimated as follows:

$$BL = \left(\frac{f}{f_1}\right)^2 B_1L_1 + \left(\frac{f}{f_2}\right)^2 B_2L_2 \qquad (4)$$

where the subscript 1 refers to the joist or beam system on rigid supports and the subscript 2 refers to the girder system.

F8.3 For floor spans less than 7000 mm, the deflection limitations given in Clause 6.2.1.2 in this Standard are recommended, where, for non-composite construction, stiffness should be based on non-composite action. In any case, care should be taken to avoid low damping.

F9. Light Wood Deck Floors Using Steel Joists

F9.1 Transient vibrations may be objectionable for light wood deck floors using steel joists with small rolled or cold formed sections, generally with frequencies in the range 10-25 Hz. Although the same principles applying to long span floors can be used for lighter floors with higher frequencies, the motion can no longer be represented by a simple impulse applied to the floor system. This is because the persons involved - the one causing and the one receiving the motion interact with the floor to damp out the motion of the floor.

F9.2 Research carried out so far on steel joist floors with wood deck indicates that, in general, their characteristics for vibration acceptability are similar to those for wood joist floors. Evaluation tests of wood floors indicate that stiffness under point loading (approximately 1 mm maximum deflection under 1 kN) is the most important parameter affecting vibration comfort[6]. Such a stiffness requirement also helps to prevent cabinet swaying, china rattling, etc. Until research under way provides a more suitable criterion, a joist deflection limitation of L/360 under 2 kPa loading is recommended. This criterion applies only when sufficient lateral stiffness is provided either in the deck or by cross-bridging.

F9.3 Floor damping is less important for light floors than for long span floors since the main source of damping is provided by the persons on the floor. Also adding mass does not improve vibration comfort since an increase in mass corresponds to a decrease in effective damping. Spans continuous over a support which is a party wall between housing units should be avoided, since people are more annoyed by vibrations originating outside their units than from within. For cold formed C joists, ceiling boards or straps should be attached to the bottom flange to prevent annoying high frequency torsional vibrations in the joists.

F10. Corrective Measures for Unacceptable Floors

F10.1 Measures for correcting floors with annoying vibrations will depend on whether the vibrations are continuous or transient.

F10.2 For transient vibrations usually the most effective measure is to increase the damping. This can be done by adding partitions or damper posts in the floor below. If these methods are not suitable, special devices such as vibration absorbers or damping materials can be incorporated into the floor system[7,8]. For light floors a rug is effective in reducing walking impact as well as in cushioning the sway of china cabinets.

F10.3 Corrective measures for continuous vibrations include vibration isolation, smoothing of road surface and alteration of floor frequency to reduce resonance.

F11. References

(1) Supplement to the National Building Code of Canada 1985. Commentary on Serviceability Criteria for Deflections and Vibrations.

(2) Thomson, W.T. "Vibration Theory and Applications", Prentice-Hall.

(3) Allen, D.E. and H. Rainer. "Vibration Criteria for Long Span Steel Floors". Can. J. Civ. Eng., Vol. 3, No. 2, June 1976.

(4) Steffens, R.J. "Some Aspects of Structural Vibration". Building Research Current Paper Engineering Series 37, Building Research Station, Ministry of Technology, Great Britain.

(5) Allen, D.L. "Vibrational Behaviour of Long-Span Floor Slabs". Proceedings of the Canadian Structural Engineering Conference, 1974.

(6) Onysko, D.M. "Performance of Wood-Joist Floor Systems - A Literature Review". Forest Products Laboratory Information Report OP-X-24, Canadian Forestry Service, Department of Fisheries and Forestry, January 1970.

(7) Nelson, F.C. "The Use of Viscoelastic Material to Damp Vibrations in Buildings and Large Structures". AISC Engineering Journal, Vol. 5, No. 2, April 1968, p. 72.

(8) Allen, D.E. and G. Pernica "Tuned Dampers for Walking Vibrations". Proceedings of 1983 Annual CSCE Conference (Submitted to Can. J. Civ. Eng.).

(9) Lenzen, K.H. "Vibration of Steel Joists-Concrete Slab Floors". AISC Engineering Journal, Vol. 3, No. 3, July 1966, p. 133.

(10) Wright, D.T. and R. Green. "Human Sensitivity to Vibrations". Department of Civil Engineering, Report No. 7, Queen's University, February 1959.

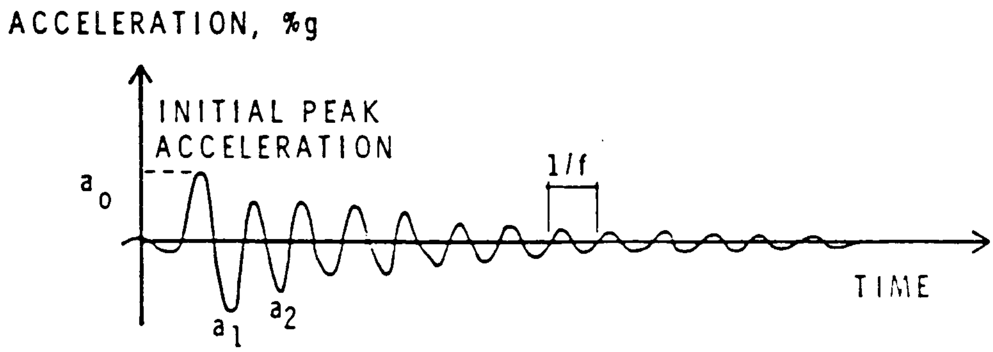


Figure F1

Typical Transient Vibration From Heel Drop (High Frequencies Filtered Out)

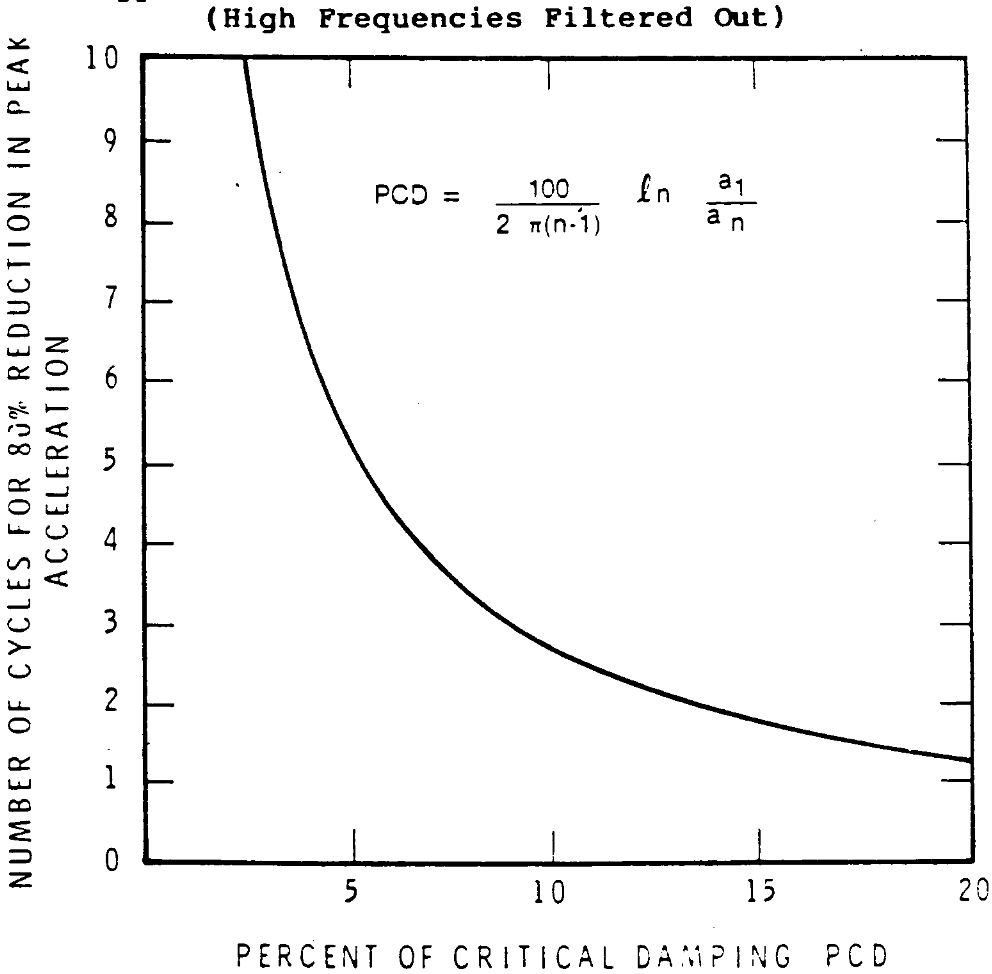


Figure F2

Relation Between Damping and Decay

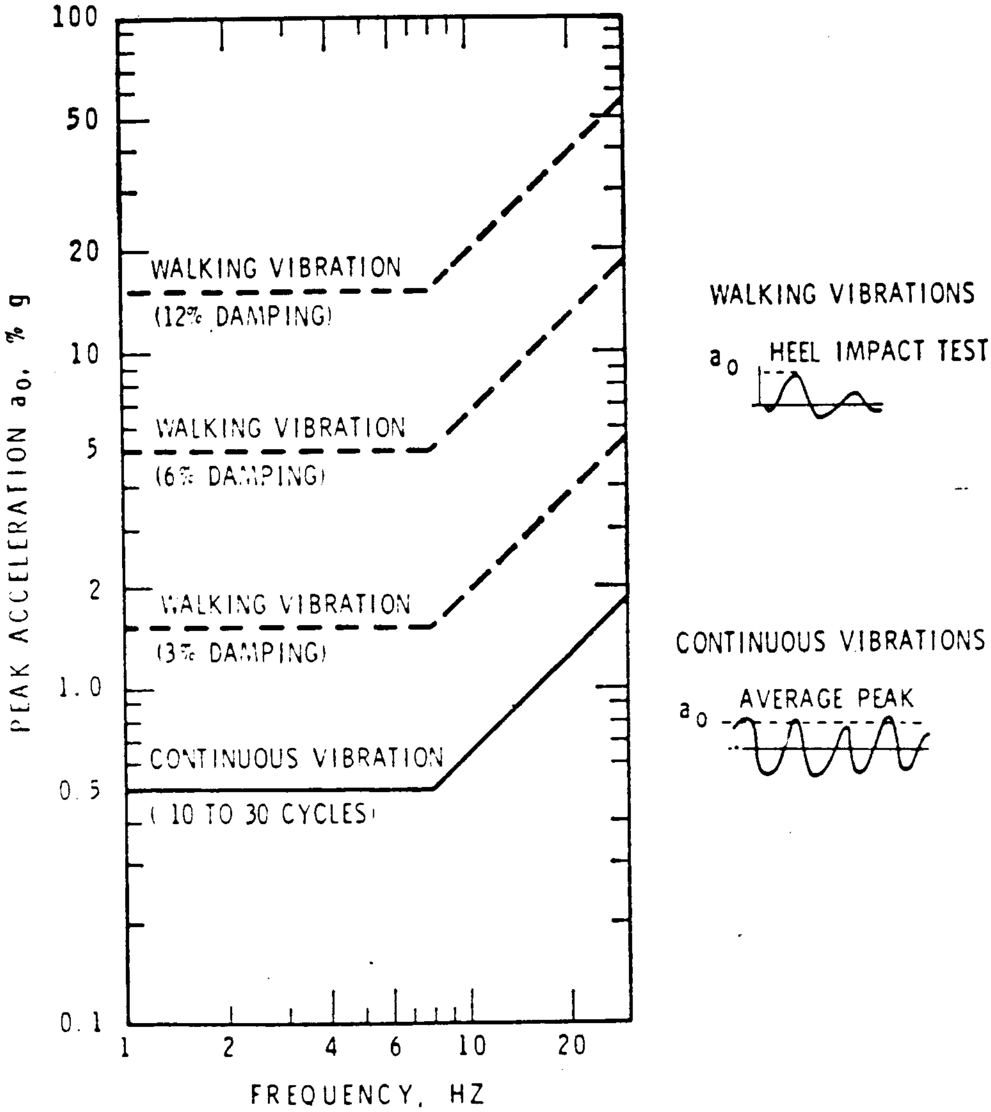


Figure F3

Annoyance Thresholds for Floor Vibrations Due to Footstep (Residential, School, Office Occupancies)

Appendix G

Wind Sway Vibrations

Note: This Appendix is not a mandatory part of this Standard.

G1. Wind motion of tall buildings or other flexible structures may create annoyance for human occupants, unless measures are taken at the design stage. The main source of annoyance is lateral acceleration, although noise (grinding and wind howl) and visual effects can also cause concern.

G2. For a given wind speed and direction, the motion of a building, which includes vibration parallel and perpendicular to the wind direction and twist, is best predicted by a wind tunnel test. Approximate calculation rules are, however, given in References 1 and 4 of Clause G4.

G3. In cases where wind motion is significant in design, the following should be considered:

(a) Education of occupants that although high winds may occasionally cause motion, the building is safe;

(b) Minimization of noises - detailing of building joints to avoid grinding, design of elevator guides to avoid scraping due to sway;

(c) Minimization of twist by symmetry of layout, bracing or outer walls (tube concept). Twist vibration also creates a magnified visual effect of relative motion of adjacent buildings;

(d) Possible introduction of mechanical damping to reduce wind vibration.

G4. **References**

(1) Supplement No. 4 to the National Building Code of Canada, 1977, Commentary on Wind Loads.

(2) Hansen, R.J., Reed, J.W. and Van Marcke, E.H. Human Response to Wind-Induced Motion of Buildings, Journal of the Structural Division, ASCE, Vol. 99, No. ST7, July 1973, p. 1589-1605.

(3) Chen, P.W. and Robertson, L.E. Human Perception Thresholds of Horizontal Motion. Journal of the Structural Division, ASCE, Vol. 98, No. ST8, August 1972, p. 1681-1695.

(4) Reed, J.W. Wind-Induced Motion and Human Discomfort in Tall Buildings. Department of Civil Engineering Research Report R71-42.

Massachusetts Institute of Technology, November 1971.

(5) Hogan, M. The Influence of Wind on Tall Building Design. Faculty of Engineering Science Research Report BLWT-4-71, University of Western Ontario, March 1971.

(6) Council on Tall Buildings and Urban Habitat. Monograph on the Planning and Design of Tall Buildings. Volumes PC and SB. American Society of Civil Engineers 1981.

Appendix H

Recommended Maximum Values for Deflections

Note: This Appendix is not a mandatory part of this Standard.

For Specified Design Live and Wind Loads*

		Due to:		
INDUSTRIAL TYPE BUILDINGS	Vertical Deflection	Live Load	Simple span members supporting inelastic roof coverings........................	$\frac{1}{240}$ of span
		Live Load	Simple span members supporting elastic roof coverings........................	$\frac{1}{180}$ of span
		Live Load	Simple span members supporting floors.....................	$\frac{1}{300}$ of span
		Maximum Wheel Loads (no impact)	Simple span crane runway girders for crane capacity of 225 kN and over.....	$\frac{1}{800}$ of span
		Maximum Wheel Loads (no impact)	Simple span crane runway girders for crane capacity under 225 kN...........	$\frac{1}{600}$ of span
	Lateral Deflection	Crane Lateral Force	Simple span crane runway girders..............................	$\frac{1}{600}$ of span
		Crane Lateral Force OR Wind	Building column sway†.................	$\frac{1}{400}$ to $\frac{1}{200}$ of height

ALL OTHER BUILDINGS	Vertical Deflection	Live Load	Simple span members of floors and roofs supporting construction and finishes susceptible to cracking..............	$\frac{1}{360}$ of span
		Live Load	Simple span members of floors and roofs supporting construction and finishes not susceptible to cracking..............	$\frac{1}{300}$ of span
	Lateral Deflection	Wind	Building sway, due to all effects............................	$\frac{1}{400}$ of bldg. height
		Wind	Storey drift, (relative horizontal movement of any two consecutive floors due to the shear effects) in buildings with cladding and partitions without special provision to accommodate building frame deformation..........	$\frac{1}{500}$ of storey height
		Wind	The same, with such special provision.........................	$\frac{1}{400}$ of height

* Although this Appendix refers specifically to Specified Design Wind and Live Loads when setting forth deflection criteria, the designer should consider the inclusion of Specified Dead Loads in some instances. For example, non-permanent partitions, which are classified by the National Building Code as dead load, should be part of the loading considered under Appendix H if they are likely to be applied to the structure after the completion of finishes susceptible to cracking. Because some building materials augment the rigidity provided by the steelwork, the wind load assumed carried by the steelwork, for calculating deflections can be somewhat reduced from the design wind used in strength and stability calculations. The more common structural elements contributing to the stiffness of a building are masonry walls, certain types of curtain walls, masonry partitions and concrete around steel members. The **maximum** suggested amount of this reduction is 15 per cent. In tall and slender structures (height greater than **4 times** the width) it is recommended that the wind effects be determined by means of dynamic analysis, or wind tunnel tests.

† Permissible sway of industrial buildings varies considerably depending on factors such as wall construction, building height, effect of deflection on the operation of crane, etc. Where the operation of the crane is sensitive to the lateral deflections, a permissible lateral deflection less than 1/400 of the height may be required.

Appendix I

Guide to Calculation of Stability Effects

Note: This Appendix is not a mandatory part of this Standard.

I1. General

I1.1 This Appendix gives one approach to the calculation of the additional bending moments and forces generated by the vertical loads acting through the deflected shape of the structure. By this approach the above moments and forces are incorporated into the results of the analysis of the structure; alternatively a second order analysis, which formulates equilibrium on the deformed structure, may be used to include the stability effects.

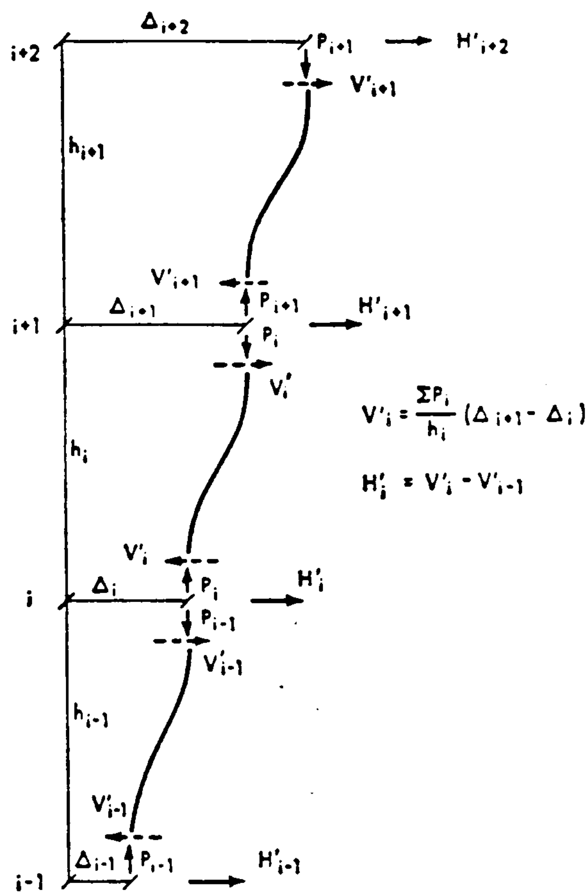


Figure I1

Sway Forces Due to Vertical Loads

I2. **Combined Loading Case**

I2.1 Step 1 - Apply the factored load combination to the structure (see Clause 7.2.2).

Step 2 - Compute the lateral deflections at each floor level (Δ_i) by first order elastic analysis

Step 3 - Compute the artificial storey shears V_i' due to the sway forces.

where

V_i' $= \frac{\Sigma P_i}{h_i}(\Delta_{i+1} - \Delta_i)$

= artificial shear in storey i due to the sway forces

ΣP_i = sum of the column axial loads in storey i

h_i = height of storey i

Δ_{i+1}, Δ_i = displacements of level i + 1 and i, respectively

Step 4 - Compute the artifical lateral loads H_i'

$$H_i' = V_{i-1}' - V_i'$$

Step 5 - Repeat Step 1 applying the artificial lateral loads H_i' in addition to the factored load combination

Step 6 - Repeat Steps 2 through 5 until satisfactory convergence is achieved. Lack of convergence within 5 cycles may indicate an excessively flexible structure.

I3. **Vertical Loads Only**

I3.1 Because vertical loads do not normally produce significant sway deflections of the structure the initial sway forces are computed on the basis of the sway displacements in each storey equal to the erection tolerance permitted by Clause 28.7.1. Using these deflections the calculations are commenced at Step 3 of the procedure described in Clause I2.1.

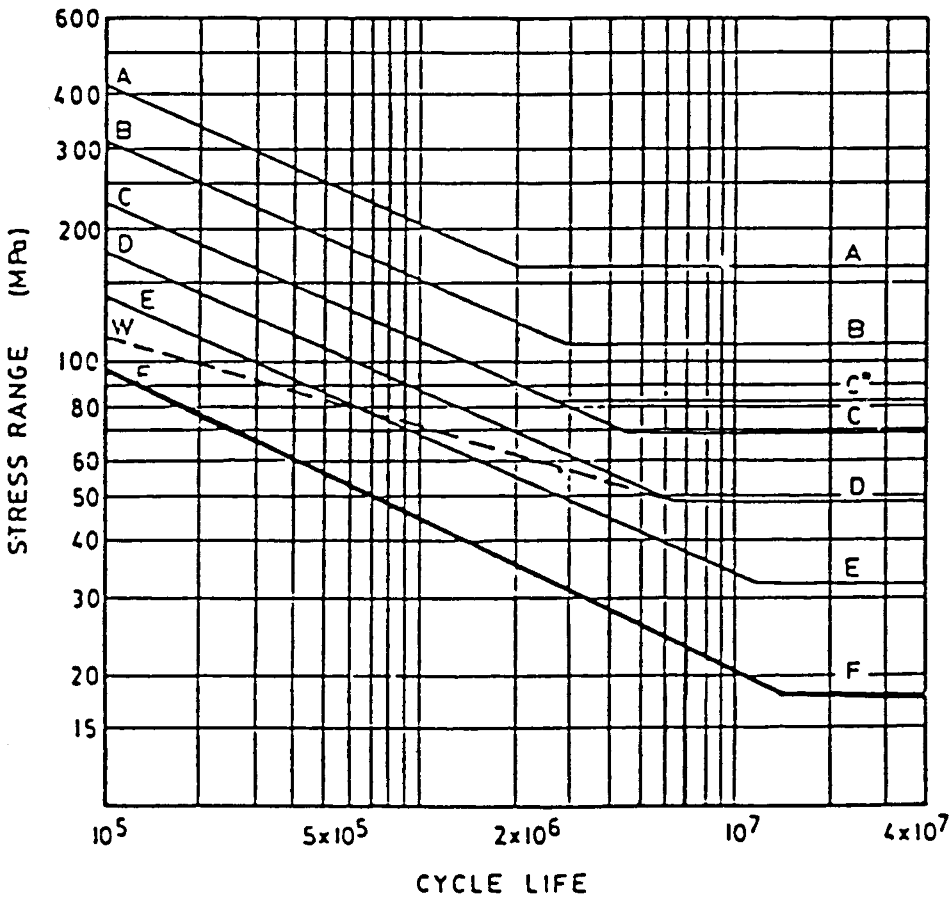
Appendix J

Fatigue

Note: This Appendix is not a mandatory part of ths Standard.

J1. Figure J1 is a plot of the design curves for the allowable stress range for categories A to F of Tables 11(a) and (b).

J2. Figure J2 gives illustrative examples of the various fatigue categories described in Table 11(b).



**Except for transverse stiffener welds on girder webs or flanges where 83 M Pa should be used.*

Figure J1

Design Curves for the Allowable Stress Range for Categories A to F

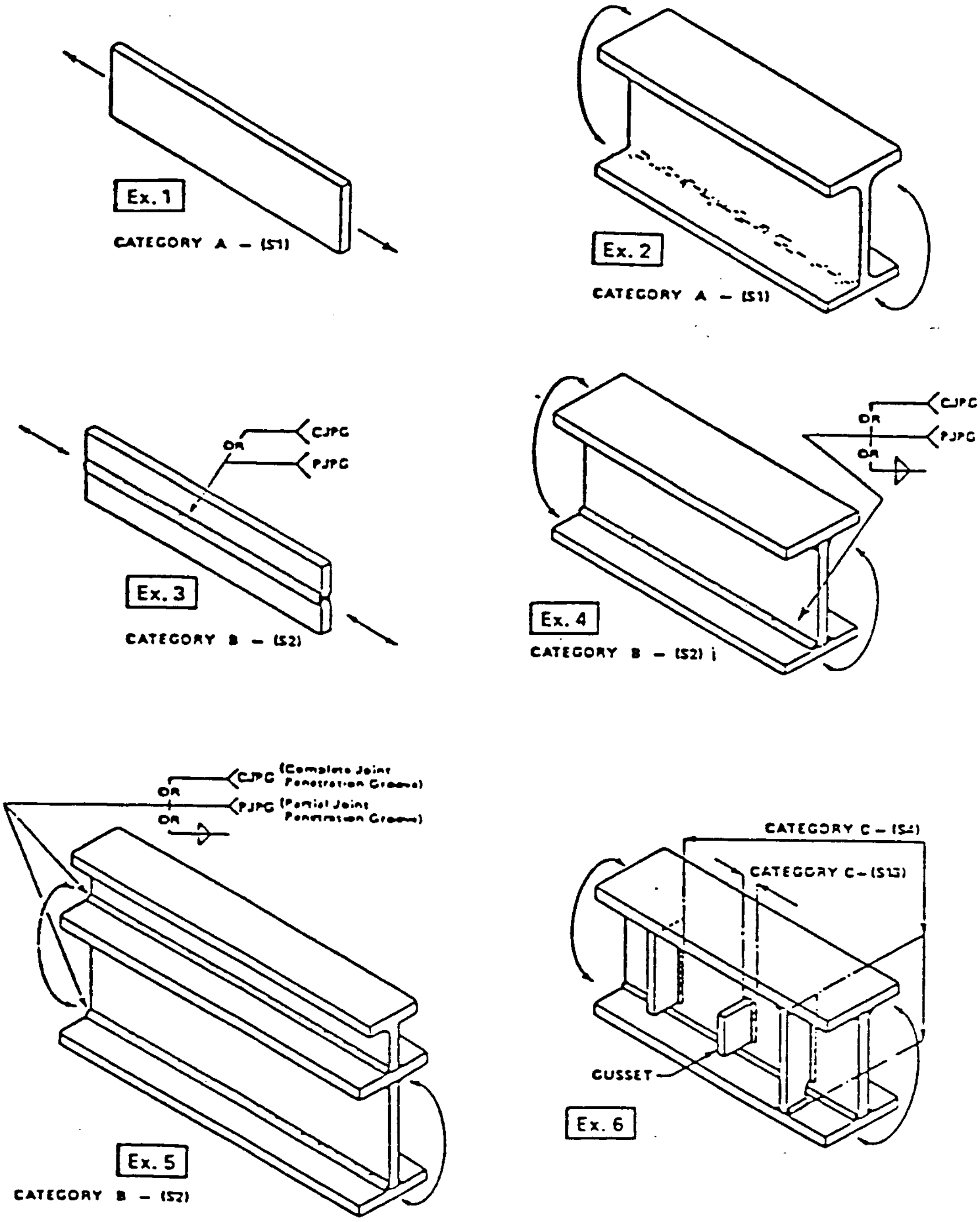


Figure J2
Illustrative Examples of Various Details
Representing Stress Range Categories

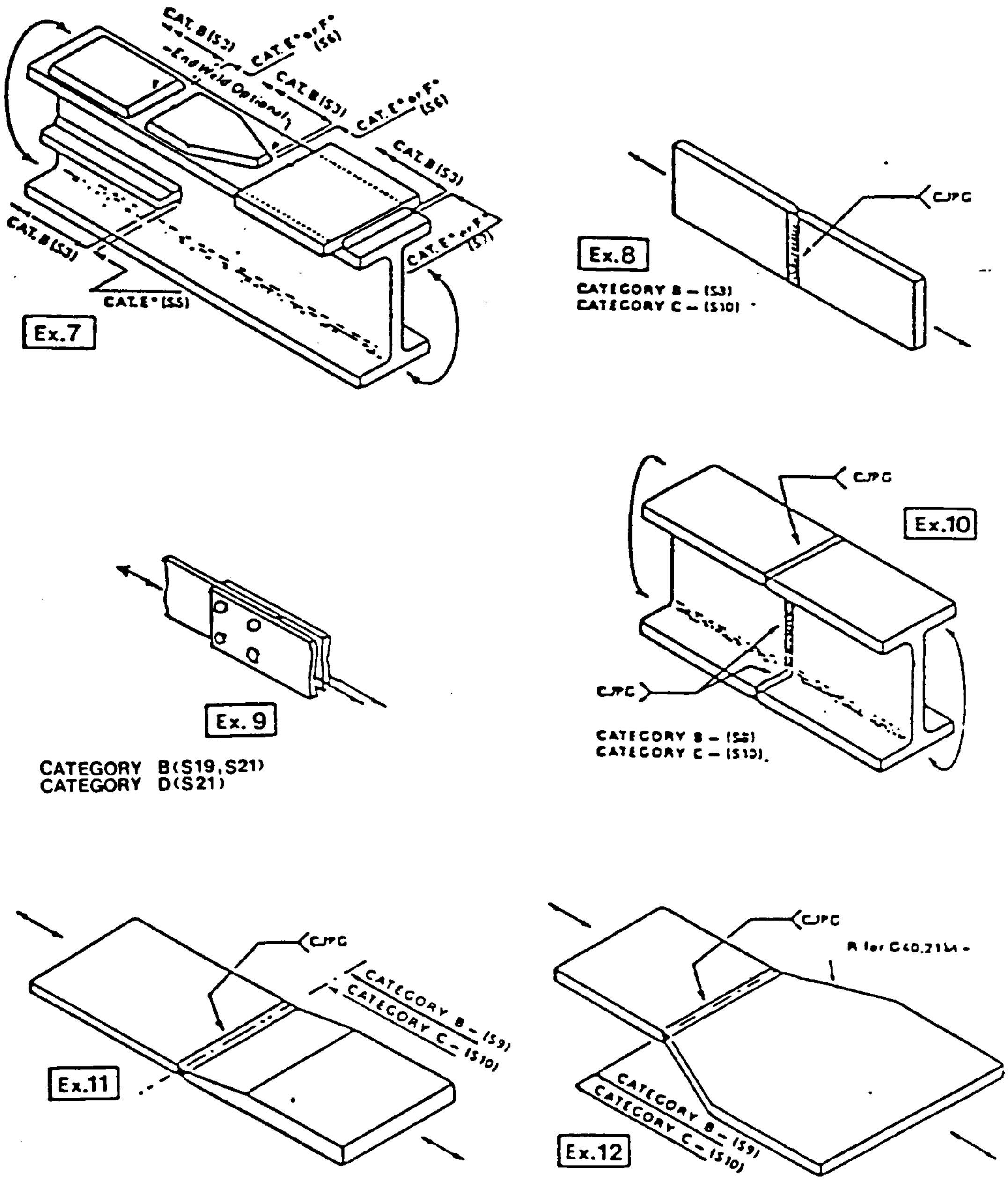


Figure J2
Illustrative Examples of Various Details
Representing Stress Range Categories (cont'd)

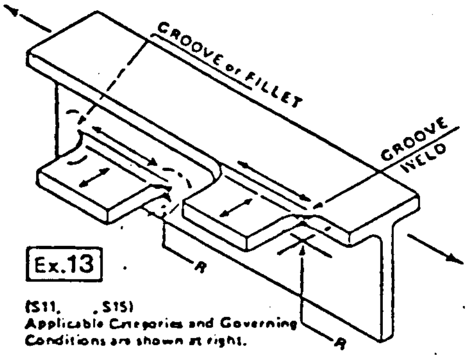
Ex. 13	Fillet Connections	Groove Connections				
	To Web	To Web To Flange	To Web (3)	To Flange (3)		
	Longitudinal Loading/Transverse Loading	Longitudinal Loading	Transverse Loading			
Transition Radius "R" (millimetres)	Stress Range Category		Stress Range Category	Stress Range Category based on Condition of Joint (2)		
				1	2	3,4
50 > R > 0	E	E(1)	E	E	E	E
150 > R > 50(4)	D	D	D	D	D	E
600 > R > 150(4)	D	C	C	C	C	E
R > 600(4)	E	B	C	B	C	E

1. For longitudinal loading only, use Category D if detail length is between 50 mm and 12 times the plate thickness, but less than 100 mm.

2. Condition of Joint:
   (1) Equal thickness of parts joined - reinforcement removed.
   (2) Equal thickness of parts joined - reinforcement not removed.
   (3) Unequal thickness of parts joined - reinforcement removed.
   (4) Unequal thickness of parts joined - reinforcement not removed.

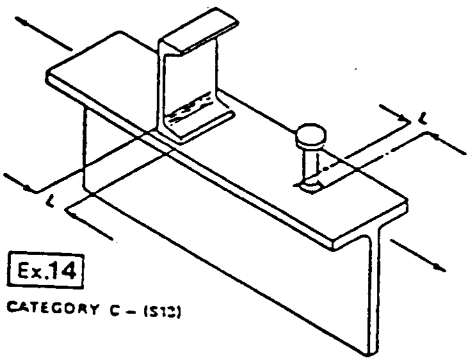
3. Weld soundness to be established by nondestructive examination.

4. Terminal ends of welded joints to be ground smooth.



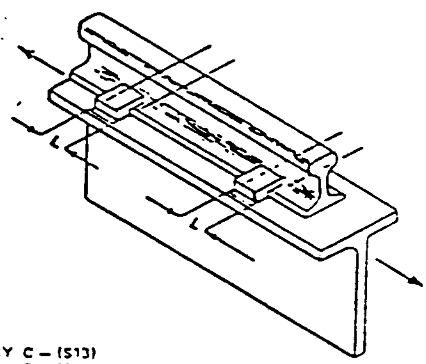
Ex.13

(S11, .S15)
Applicable Categories and Governing Conditions are shown at right.



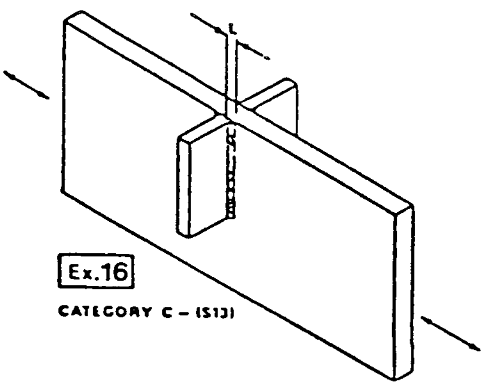
Ex.14

CATEGORY C – (S12)



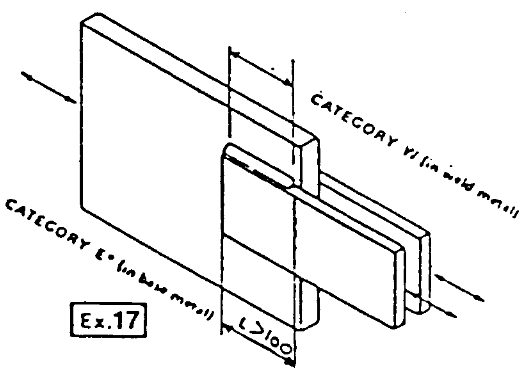
Ex.15

CATEGORY C – (S13)
CATEGORY D – (S14)



Ex.16

CATEGORY C – (S13)



Ex.17

Figure J2
Illustrative Examples of Various Details
Representing Stress Range Categories (cont'd)

Appendix K

Deflections of Composite Beams Due to Shrinkage Strain

Note: This Appendix is not a mandatory part of this Standard.

K1.1 During the curing of the concrete cover slab, the volumetric reduction (shrinkage strain) of the concrete induces additional flexural deflections in the composite member when the concrete slab contracts but the steel shape does not.

K1.2 Two models have been suggested[1,2] to estimate the deflection due to shrinkage. Basically, these models assume a value of shrinkage strain acting on the effective area of the concrete slab. It is assumed that the shrinkage strain is equivalent to a constant moment acting on the composite beam by selecting an appropriate modulus of elasticity for concrete and a lever arm of the force taken from the centre of the slab to the elastic neutral axis of the composite beam (Figure K.1).

K2. The resulting shrinkage deflection is:

$$\Delta_{sh} = \frac{\varepsilon_{sh} E_c A_c L^2}{8EI} \cdot y_c$$

where

ε_{sh} = the shrinkage strain
E_c = modulus of elasticity of concrete
A_c = effective area of concrete slab
L = span length of beam
E = modulus of elasticity of steel beam
I = moment of inertia of composite beam as given in K3 or K4
y_c = distance from elastic neutral axis to assumed line of action of the shrinkage face.

K3. The first method[1] uses the restrained shrinkage value for ε_{sh}, which is lower than the free shrinkage strain, the normal modulus of elasticity, E_c, and the transformed moment of inertia, I_t.

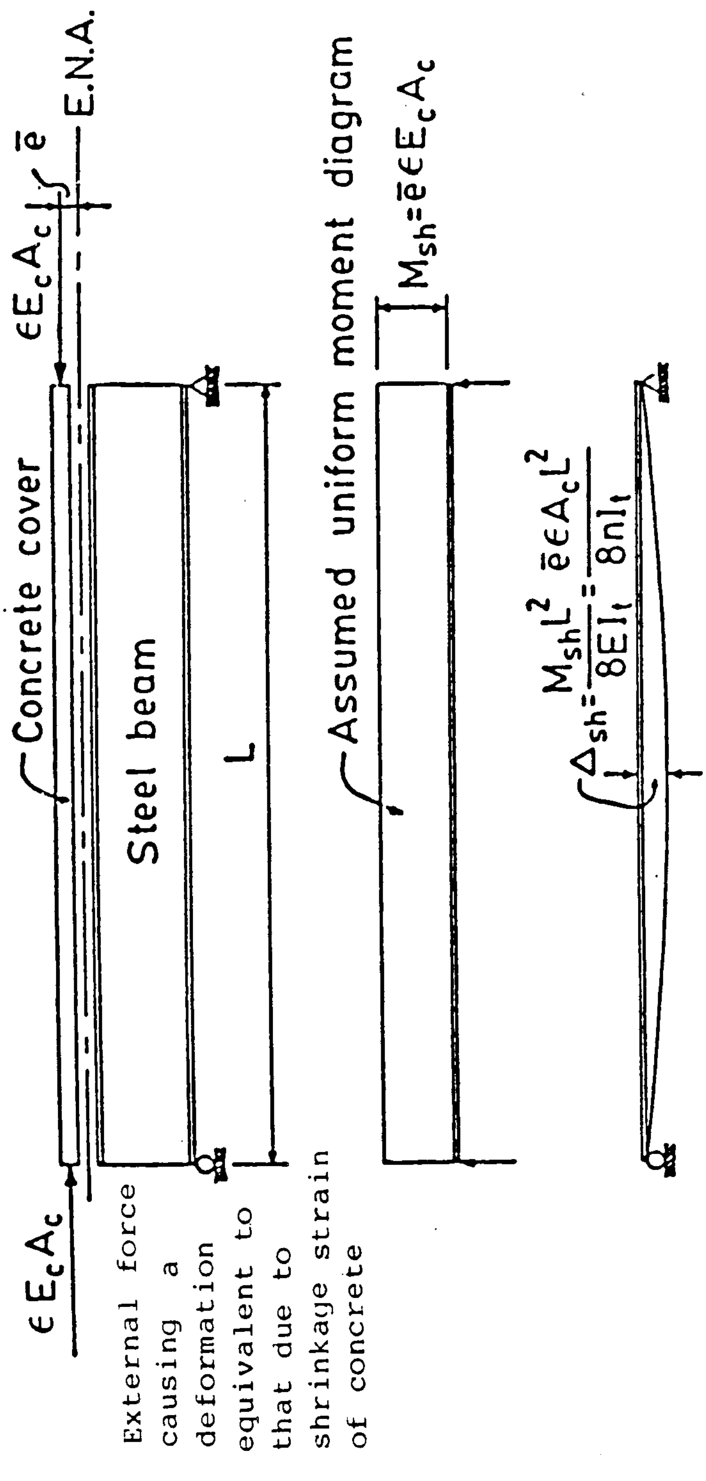
K4. The second method[2] takes the shrinkage strain as the strain appropriate to free shrinkage and reflects the influence on the free shrinkage of the following: the time over which shrinkage occurs, relative humidity, volume-to-surface ratio, slump, fires, air contact and cement contact of the concrete mix. An age-adjusted modulus, similar to that used in calculations of concrete creep, is used for E_c and the effective moment of inertia I_e.

K5. For both methods, care should be taken in selecting values for the quantities in equation K.1 so as to neither over or under estimate deflections especially when used to determine the serviceability requirements. Many of the quantities in equation K.1 will be influenced by site conditions.

K6. **References:**

(1) Chien, E.Y.L., "Composite Floor Systems" (to be published by Canadian Institute of Steel Construction).

(2) Montgomery, C.J., Kulak, G.L., and Shwartsburd, G., "Deflection of a Composite Floor System", Can. J. Civ. Eng., Vol. 10, No. 2, June, 1983.



E.N.A.
$\bar{e}$
$\epsilon E_c A_c$
$\epsilon E_c A_c$
Concrete cover
Steel beam
L
External force causing a deformation equivalent to that due to shrinkage strain of concrete
$M_{sh} = \bar{e}\epsilon E_c A_c$
Assumed uniform moment diagram
$\Delta_{sh} = \frac{M_{sh}L^2}{8EI_t} = \frac{\bar{e}\epsilon A_c L^2}{8nI_t}$

Figure K1

Appendix L

Columns Subject to Biaxial Bending

Notes:

(1) This Appendix is not a mandatory part of this Standard.

(2) The precise design of beam-columns to resist biaxial bending is extremely complex. More refined design expressions than those given in Clause 13 are available but these are shape dependent.

L1 Class 1 and 2 wide flange shapes, may be proportioned so that:

(a) $$\left(\frac{M_{fx}}{M_{rcx}}\right)^{\zeta} + \left(\frac{M_{fy}}{M_{rcy}}\right)^{\zeta} \leq 1.0$$

(b) $$\left(\frac{\omega_x M_{fx}}{M_{ox}}\right)^{\eta} + \left(\frac{\omega_y M_{fy}}{M_{oy}}\right)^{\eta} \leq 1.0$$

Where M_{rcx}, M_{rcy} are the factored moment resistance of the section, reduced for the presence of axial load, and may be taken as:

$$M_{rcx} = 1.18 M_{rx} \left(1 - \frac{C_f}{C_y}\right) \leq M_{rx}$$

$$M_{rcy} = 1.19 M_{ry} \left[1 - \left(\frac{C_f}{C_y}\right)^2\right] \leq M_{ry}$$

Where M_{rx} and M_{ry} are defined in Clause 13.5.1(a)

M_{ox}, M_{oy} = maximum single curvature factored moment resistance of the column in the presence of the axial load but in the absence of the other orthogonal moment, which may be taken as:

$$M_{ox} = M_{rx} \left(1 - \frac{C_f}{C_r}\right)\left(1 - \frac{C_f}{C_{ex}}\right)$$

$$M_{oy} = M_{ry} \left(1 - \frac{C_f}{C_r}\right)\left(1 - \frac{C_f}{C_{ey}}\right)$$

$$\zeta = 1.6 - \frac{C_f/C_y}{2\ln(C_f/C_y)}$$

$$\eta = 0.4 + \frac{C_f}{C_y} + \frac{b}{d} \quad \text{for } \frac{b}{d} \geq 0.3$$

and

$\eta = 1.0 \qquad \text{for } \frac{b}{d} < 0.3$

where M_{rx} is defined in Clause 13.5.2(a), M_{ry} is defined in Clause 13.5.1(a) and C_r is defined in Clause 13.3.1.

Note: For values of $C_f/C_y < 0.3$, the value of ζ may be taken equal to the value of η.

L2 Class 1 and Class 2 square hollow structural sections (rolled* or fabricated) may be proportioned so that:

(a) $$\frac{M_{fx}}{M_{fx}} + 0.5\,\frac{M_{fy}}{M_{ry}} < 1.0$$

(b) $$\frac{C_f}{C_r} + 0.85\left(\frac{M_{fx}}{M_{rx}} + 0.5\,\frac{M_{fy}}{M_{ry}}\right) < 1.0$$

where

M_{fx} = the numerically larger moment

M_{rx} and M_{ry} are defined in Clause 13.5.1(a)

$C_r = \phi A F_y$

* Hot rolled or stress relieved such that residual stresses do not exceed $0.3F_y$.

(c) $$\frac{C_f}{C_r} + \nu\,\frac{\omega_x M_{fx}}{M_{rx}\left(1 - \frac{C_f}{C_{ex}}\right)} + \frac{\omega_y M_{fy}}{M_{ry}\left(1 - \frac{C_f}{C_{ey}}\right)} < 1.0$$

where $$\nu = \frac{\sqrt{(\omega_x M_{fx})^2 + (\omega_y M_{fy})^2}}{\omega_x M_{fx} + \omega_y M_{fy}}$$

C_r is defined in Clause 13.3.1

M_{rx} is defined in Clause 13.5.2(a)

M_{ry} is defined in Clause 13.5.1(a)

Note: Design of Class 1 and Class 2 sections in accordance with the above requirements takes advantages of the redistribution of stress after initiation of yielding, under the factored loads. Consideration should be given to this aspect of design if yielding under the specified loads would induce undesirable lateral deformations of a structure.

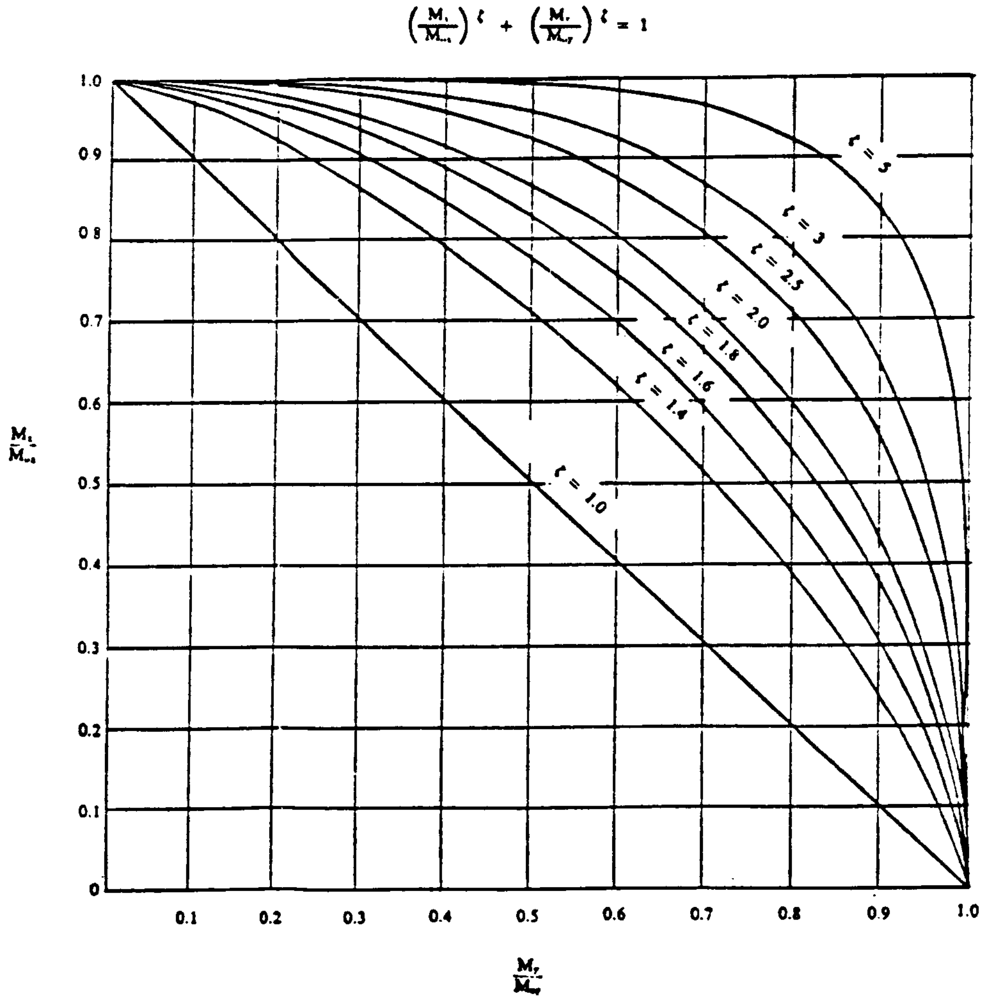


Figure L1

Plot of the Interaction Equation

FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

Feet^2 (ft^2) x 0.0929030	= Meters^2 (m^2)
Acres x 4046.86	= Meters^2 (m^2)
Miles^2 (mi^2) x 2.58999	= Kilometers^2 (km^2)

VOLUME

Feet^3 (ft^3) x 0.02831685	= Meters^3 (m^3)
Feet^3 (ft^3) x 28.31685	= Liters**
Yards^3 (yd^3) x 0.764555	= Meters^3 (m^3)
Acre feet x 1233.48	= Meters^3 (m^3)
Gallons (gal) x 0.00454609	= Meters^3 (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

Feet^3 per second (ft^3/s) x 0.02831685	= Meters^3 per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= Meters^3 per second (m^3/s)
Acre-feet per day x 0.0142764	= Meters^3 per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= Meters^3 per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 (lb-s/ft^2) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

---

* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 7B

Structural Design Requirements
STRUCTURAL STEEL

WORKING STRESS DESIGN

1985

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 7B
STRUCTURAL STEEL

WORKING STRESS DESIGN

Caribbean Community Secretariat
Georgetown
Guyana

1985

PART 2

SECTION 7

TABLE OF CONTENTS

FOREWORD

This Section is divided into three parts:

Section 7A - Limit States Design
Section 7B - Working Stress Design
Section 7C - Commentary

The work on this Section was carried out by Messrs. Adams, Kennedy and Kulak of the Department of Civil Engineering, University of Alberta, under a contract with the Caribbean Community. It is suggested that comments on the alternative design methods, or on any of the design details recommended, be sent to the authors.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

Each of the Sections 7A, Limit States Design; 7B, Working Stress Design; and 7C, Commentary have been numbered individually so as to provide continuity between sections. The number and digit corresponding to the Part and Section in the Part (2.7) have been omitted. The numbers that remain correspond to the sub-sections and articles.

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 7B

STRUCTURAL STEEL

WORKING STRESS DESIGN

Disclaimer

The authors of this publication, Messrs. Adams, Kennedy and Kulak, do not warrant the contents and the suitability of this material for general or particular use, nor is there any implication that there is freedom from infringement of patents. The design of structures is within the scope of expertise of a competent licensed professional. Such user of this Standard assumes all liability arising from its use.

Contents

Preface

This is the first edition of the Standard, "Structural Steel for Buildings", Part 2.7B, Working Stress Design, developed for the Caribbean Uniform Building Code. A first edition of a parallel Standard Steel Structures for Buildings, Part 2.7A, Limit States Design has also been produced for the Caribbean Uniform Building Code. The third document in this sequence is Steel Structures for Buildings Part 2.7C - Commentary.

The Standard is based on the working stress design philosophy. In working stress design the designer compares working stresses, determined from the effect of specified loads, with allowable stresses set at some portion of the stresses associated with the maximum strength of the member. In general a single factor of safety equal to the ratio of the "failure stress" to the working stress is used. For this Standard the value is generally 1.67. This approach does not take into account the variability of the different loads and resistance, as is the case in limit states design. The result therefore, is a greater variation in safety of the structure and less economy.

The clauses relating to fabrication and erection should serve to remind designers that design and construction are part of the same sequence. The review of construction by competent engineers is of equal importance to the design.

When designing structures under this Standard no use shall be made of the companion standard, Steel Structures for Buildings - Limit States Design. The Commentary clarifies the intent of various provisions of this Standard. The publications listed as references provide the extensive background used for the development of the Standard and its technical requirements.

This Standard sets out minimum requirements for steel structures as outlined in the Scope, and it is expected that it will only be used by engineers competent in the field. Although the intended primary application of the Standard is stated in the Scope clause, it is important to note that it remains the responsibility of the user of the Standard to judge its suitability for his particular purpose.

The Standard is based in large measure on CSA Standard CAN3-S16.1-M84 Steel Structures for Buildings - Limit States Design and on an October 1983 draft of CSA Standard CAN3-S136-M84 Cold Formed Steel Structural Members and reflects the latest research on steel building structures. It has, however, where appropriate, been couched in working stress design format. The Canadian Standards Association assumes no responsibility for its content.

Although every effort has been made in writing and proofreading this Standard to ensure that all information is accurate and that all numerical values are correct some errors may have been overlooked. Users are requested to bring any such errors found to the attention of the authors.

Reference Publications

This Standard refers to the following publications. The years shown indicate the latest issues available at the time of printing.

CSA Standards

CAN3-A23.1-M77,
Concrete Materials and Methods of Concrete Construction;

CAN3-A23.1-M84,
Code for the Design of Concrete Structures for Buildings;

B95-1962,
Surface Texture (Roughness, Waviness, and Lay);

G28-1968,
Carbon-Steel Castings for General Application;

G38-1953,
Heavy Steel Shaft Forgings;

CAN3-G40.20-M81,
General Requirements for Rolled or Welded Structural Quality Steel;

CAN3-G40.21-M81,
Structural Quality Steels;

G189-1966,
Sprayed Metal Coatings for Atmospheric Corrosion Protection;

S37-M1981,
Antenna Towers and Antenna Supporting Structures;

S136-M1984,
Cold Formed Steel Structural Members;

W47.1-1973,
Certification of Companies for Fusion Welding of Steel Structures;

W48.1-M1980,
Mild Steel Covered Arc-Welding Electrodes;

W48.3-1976,
Low-Alloy Steel Arc-Welding Electrodes;

W48.4-M1980,
Solid Mild Steel Electrodes for Gas Metal-Arc Welding;

W48.5-1970,
Mild Steel Electrodes for Flux Cored Arc Welding;

W48.6-M1980,
Bare Mild Steel Electrodes and Fluxes for Submerged-Arc Welding;

W55.3-1965,
Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings;

W59-1977,
Welded Steel Construction (Metal-Arc Welding);

ANSI/ASTM† Standards
A27-80,
Mild- to Medium-Strength Carbon-Steel Castings for General Application;

A36-77a,
Structural Steel;

A108-79,
Steel Bars, Carbon, Cold-Finished, Standard Quality;

A148-80,
High Strength Steel Castings for Structural Purposes;

A242-81,
High-Strength Low-Alloy Structural Steel;

A243-81,
Low and Intermediate Tensile Strength Carbon Steel Plates of Structural Quality;

A307-80,
Carbon Steel Externally and Internally Threaded Standard Fasteners;

A325-82,
High-Strength Bolts for Structural Steel Joints;

A325 M-82,
High-Strength Bolts for Structural Steel Joints (Metric);

A441-79,
High-Strength Low-Alloy Structural Manganese Vanadium Steel;

A446-76,
Steel Sheet, Zinc Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality (Grades A, B, C, D and F);

A486-74 (Reapproved 1980),
Steel Castings for Highway Bridges;

A490-82,
Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength;

A490 M-82,
High-Strength Steel Bolts, Classes 10.9 and 10.9.3 for Structural Steel Joints (Metric);

A514-77,
High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding;

A521-76,
Steel, Closed-Impression Die Forgings for General Industrial Use;

A525-79,
Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, General Requirements;

A570-79,
Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality;

A572-81a,
High-Strength Low-Alloy Columbium-Vandium Steel of Structural Quality;

A588-80a,
High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick;

A606-75,
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength, Low-Alloy, with Improved Corrosion Resistance;

A607-75,
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy Columbium and/or Vanadium;

A611-72,
Steel, Cold-Rolled Sheet, Carbon, Structural (Grades A, B, C and D);

A668-79a,
Steel Forgings, Carbon and Alloy, for General Industrial Use;

A715-81,
Sheet Steel and Strip, Hot-Rolled, High-Strength Low-Alloy, with Improved Formability;

CGSB# Standards
1-GP-14M-1979,
Primer, Red Lead in Oil;

1-GP-40M-1979,
Primer, Structural Steel, Oil Alkyd Type;

1-GP-81M-1978,

Primer, Alkyd, Air Drying and Baking, for Vehicles and Equipment;

1-GP-140M-1978,
Primer, Red Lead, Iron Oxide, Oil Alkyd Type;

1-GP-166M-1979,
Primer, Basic Lead Silico-Chromate, Oil Alkyd Type;

CISC/CPMA§ Standards
1-73a,
A Quick-Drying One-Coat Paint For Use On Structural Steel;

2-75,
A Quick-Drying Primer For Use On Structural Steel;

Canadian Institute of Steel Construction
Code of Standard Practice for Structural Steel;

CSSBI†† Standards
101 M-78,
Zinc Coated Structural Quality Steel Sheet for Roof and Floor Deck;

National Building Code of Canada, 1980;

Metric Values for Use with the National Building Code, 1980;

SSPC Specifications**
PS 12.00-68T,
Guide to Zinc-Rich Coating Systems;

PT 3-64,
Basic Zinc Chromate - Vinyl Butyral Washcoat;

SP 2-63,
Hand Tool Cleaning;

SP 3-63,
Power Tool Cleaning;

SP 4-63,
Flame Cleaning of New Steel;

SP 5-63,
White Metal Blast Cleaning;

SP 6-63,
Commercial Blast Cleaning;

SP 7-63,
Brush-Off Blast Cleaning;

SP 10-63T,
Near-White Blast Cleaning;

Structural Stability Research Council
Guide to Stability Design Criteria for Metal Structures.

* American National Standards Institute.
† American Society for Testing and Materials.
‡ Canadian General Standards Board.
§ Canadian Institute of Steel Construction/Canadian Paint Manufacturers' Association.
†† Canadian Sheet Steel Building Institute
** Steel Structures Painting Council.

Structural Steel - Working Stress Design

Where reference in this Standard is made to National Standards of Canada, National Standards of other countries may be used when approved by the Regulatory Authority.

1. **Scope**

1.1 This Standard provides rules and requirements for the design, fabrication, and erection of steel structures for buildings where the design is based on working stress procedures. The term "steel structures" relates to structural members and frames which consist primarily of structural steel components, including the detail parts, welds, bolts, or other fasteners required in fabrication and erection. Composite construction, defined as construction which depends upon the participation of structural elements of steel and other materials in resisting loads and forces jointly with structural steel members, is permitted under this Standard. Clause 14 applies to the design of structural members cold formed to shape from carbon or low alloy steel sheet, strip or plate up to 25 mm in thickness and intended for load carrying purposes in buildings.

1.2 Where reference is made to other publications, such reference shall be considered to refer to the latest edition or any revision thereto approved by the organization issuing that publication.

1.3 When designing structures under this Standard, no use shall be made of the companion Standard, Steel Structures for Buildings - Limit States Design.

2. **Application**

2.1 This Standard applies unconditionally to steel structures for buildings except as noted in Clause 2.2.

2.2 Supplementary rules or requirements may be necessary for:

(a) Unusual types of construction

(b) Mixed systems of construction; and

(c) Steel structures which

(i) Have great height or spans;

(ii) Are required to be movable or be readily dismantled;

(iii) Are exposed to severe environmental conditions, or possible severe loads such as those resulting from vehicle impact or chemical explosion;

(iv) Are required to satisfy aesthetic, architectural, or other requirements of a non-structural nature;

(v) Employ materials or products not listed in Clause 5;

(vi) Have other special features that could affect design, fabrication, or erection.

2.3 A rational design based on theory, analysis, and engineering practice, acceptable to the Regulatory Authority, may be used in lieu of the formulae provided in this Standard. In such cases the design shall provide nominal margins (or factors) of safety at least equal to those intended in the provisions of this Standard (see Appendix E).

3. Definitions and Symbols

3.1 Definitions

The following definitions apply to this Standard:

General

Approved means approved by the Regulatory Authority;

Regulatory Authority means a government Ministry, Department, Board, Agency, or Commission that has responsibility for regulating, by statute, the use of products, materials, or services;

Working stress design means that design procedure in which stresses in members due to the specified loads are held to allowable values, determined to be a fraction of the limiting stress. The limiting stress may be based on such conditions as yielding, fatigue, buckling or fracture.

Plastic design means that design procedure in which the behaviour of the structure under the action of factored loads is determined by taking into account the plastic behaviour of the members and resulting plastic redistribution of moments and forces in the structure.

Loads

Gravity load (newtons) is equal to the mass of the object (kilograms) being supported multiplied by the acceleration due to gravity, g (9.81 m/s^2);

Specified loads (D, L, Q and T) means those loads prescribed by the Regulatory Authority (see Clause 7.1);

Factors

Load factor, α, means a factor, given in Clause 31, applied to a specified load for the plastic method of design to take into account the variability of the loads and load patterns and analysis of their effects;

Load combination factor, ψ, means a factor, given in Clause 7.2, applied to the loads to take into account the reduced probability of a number of loads from different sources acting simultaneously;

Tolerances

Camber means the deviation from straightness of a member or any portion of a member with respect to its major axis. Frequently camber is specified and produced in a member to compensate for deflections that will occur in the member when loaded. (See Clause 6.2.2.) Unspecified camber is sometimes referred to as bow;

Sweep means the deviation from straightness of a member or any portion of a member with respect to its minor axis;

Mill tolerances means variations allowed from the nominal dimensions and geometry with respect to cross-sectional area, non-parallelism of flanges and out-of-straightness such as sweep or camber in the product as manufactured and are given in CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel;

Fabrication tolerances means tolerances allowed from the nominal dimensions and geometry such as the cutting to length, finishing of ends, cutting of bevel angles, and for fabricated members, out-of-straightness such as sweep and camber (see Clause 27);

Erection tolerances means tolerances related to the plumbness, alignment, level, of the piece as a whole. The deviations are determined by considering the locations of the ends of the piece with respect to the positions stipulated on the drawings.

Note: Additional definitions are found in the Standard particularly in Clauses 14, 17 and 18.

3.2 **Symbols**

The following symbols are used throughout this Standard. Deviations from them, and additional nomenclature, are noted where they appear. Dimensions in mm and forces in newtons are assumed unless otherwise noted.

A	Area
A_b	Area term used in the calculation of capacity of bearing stiffeners
A_c	Area of concrete; area term used in the calculation of capacity of bearing stiffeners
A_f	Flange area
A_{fc}	Compression flange area
A_g	Gross area
A_m	Area of fusion face
A_n	Critical net area
A_r	Area of reinforcing steel
A_s	Area of steel section including cover plates; area of bottom (tension) chord of steel joist; area of stiffener or pair of stiffeners
A_{sc}	Area of steel shear connector
A_w	Web area; shear area; effective throat area of weld
a	Centre-to-centre distance between transverse web stiffeners; depth of concrete compression zone
a'	Length of cover plate termination
a/h	Aspect ratio, ratio of distance between stiffeners to web depth
b	Width of stiffened or unstiffened

	compression elements; design effective width for a concrete slab of a composite beam
C	Coefficient in formula for area of transverse stiffeners
D	Outside diameter of circular section; diameter of rocker or roller; also stiffener factor
d	Depth; overall depth of a section; diameter of bolt or stud; longitudinal distance centre-to-centre of battens
E	Elastic modulus of steel (200 000 MPa ksi assumed)
E_c	Elastic modulus of concrete
e	End distance
F	Allowable stress (MPa unless noted)
F_a	Allowable axial stress in compression
F'_a	Allowable axial stress on a short column = $0.60F_y$
F_b	Allowable bending stress
F_{bc}	Allowable bending stress in compression
F'_{bc}	Reduced allowable bending stress in compression
F_{bt}	Allowable bending stress in tension
F'_e	Euler buckling stress divided by factor of safety
F_p	Allowable stress in compression due to bearing
F_{sr}	Allowable range of stress in fatigue
F_t	Allowable axial stress in tension
F_u	Specified minimum tensile strength
F_v	Allowable shear stress
F_y	Specified minimum yield stress, yield point or yield strength

F_{yb}	Specified yield strength of beam steel
F_{yc}	Specified yield strength of column steel
F_{yr}	Specified yield strength of reinforcing steel
F_{ys}	Specified yield strength of stiffener steel
F_1	Allowable stress in bending
F_2	Allowable stress in bending
F_3	Allowable stress in bending
f	Computed stress
f_a	Computed axial stress in compression
f_b	Computed bending stress
f'_c	Specified compressive strength of concrete at 28 days
f_t	Computed axial stress in tension
f_v	Computed shear stress
G	Shear modulus of steel (77 000 MPa assumed)
g	Transverse spacing between fastener gauge lines (gauge distance)
h	Clear depth of web between flanges; height of stud
h_c	Clear depth of column web
I	Moment of inertia
I_e	Effective moment of inertia
I_g	Moment of inertia of cover plated section
I_s	Moment of inertia of the steel section
I_t	Moment of inertia of the transformed composite section
K	Effective length factor
KL	Effective length
k	Distance from outer face of flange to web

	toe of fillet of I-type sections; buckling coefficient for slender sections
k_b	Buckling coefficient
k_v	Shear buckling coefficient
L	Length
L_c	Length of channel shear connection
L_{cr}	Maximum unbraced length adjacent to a plastic hinge
M	Specified bending moment
M_c	Moment of point at theoretical cut off
M_{fx}	Factored moment about x-x axis
M_{fy}	Factored moment about y-y axis
M_{max}	Maximum positive bending moment
M_s	Moment capacity of the steel section alone
M_y	Yield moment = SF_y
M_1	Smaller end moment of a beam-column
M_2	Larger end moment of a beam-column
N	Length of bearing of an applied load
n	Number of shear connectors required for full composite action
n'	Number of shear connectors provided
n''	Number of shear connectors required between any concentrated load and nearest point of zero positive bending moment
P	Specified axial load; force in cover plate
P_a	Allowable axial load on a column
P_c	Allowable axial load on concrete area of composite column
P_e	Euler buckling load
P_f	Factored axial load in plastic design

P_s Allowable axial load on steel area of composite column

P_y Axial load at yield stress

p Fraction of full shear connection expressed as a decimal; pitch

q Capacity of a shear connector

R End reaction or concentrated transverse load applied to a flexural member; algebraic ratio minimum to maximum stress in fatigue

r Radius of gyration

r_c Radius of gyration of a concrete area, A_c

r_t Radius of gyration about its axis of symmetry of a tee section comprising the compression flange and 1/6 of the web (inches)

S Elastic section modulus of a steel section; short term load on a composite column

s Centre-to-centre spacing (pitch) between successive fastener holes in line of stress

T Total load on a composite column

t Thickness

t_b Thickness of beam flange

t_c Thickness of column flange

t_f Flange thickness

t_w Web thickness of channel shear connection

V Statical shear

V_f Statical shear produced by factored load in plastic design

V_h Horizontal shear to be resisted by shear connectors

w Web thickness

w_c Web thickness of a column

x Subscript relating to strong axis of member

Y	Ratio of yield stress of webs to yield stress of stiffeners
y	Subscript relating to weak axis of member; distance to extreme fibre from neutral axis
y_c	Distance from elastic neutral axis to assumed line of action of the shrinkage force
Z	Plastic section modulus of a steel section
Δ_{sh}	Shrinkage deflection
α	Value used in bolt interaction equation
β	Value used in bolt interaction equation
β'	Value used in bolt interaction equation
γ	Importance Factor; coefficient in shear formula
ε_{sh}	Shrinkage strain
η	Coefficient in shear formula
λ	Non-dimensional slenderness ratio used in column formula
μ	Coefficient related to the slip resistance of a bolted joint
ρ	Constant which depends upon Poisson's ratio for steel and concrete
τ	Empirical coefficient used to account for additional strength from a triaxial stress state developed in a concrete filled column
τ'	Empirical coefficient used to account for additional strength from a triaxial stress state developed in a concrete filled column
ψ	Load combination factor
ω	Coefficient used to determine equivalent uniform bending effect in beam-columns

4. Drawings

4.1 Design Drawings

4.1.1 Design drawings shall be drawn to a scale adequate to convey the required information. The drawings shall show a complete design of the structure with members suitably designated and located, including such dimensions and detailed description as necessary to permit the preparation of shop details and erection diagrams. Floor levels, column centres, and offsets shall be dimensioned.

4.1.2 Design drawings shall designate the design standards used, show clearly the type or types of construction as defined in Clause 8 to be employed, and shall designate the material or product Standards applicable to the members and details depicted (see Clause 5). Drawings shall be supplemented by data concerning the governing loads, shears, moments, and axial forces to be resisted by all members and their connections when needed for the preparation of shop details. (See also Clause 20.1.2.)

4.1.3 Where high-strength bolted joints are required to resist shear between connected parts, the design drawings shall indicate the type of joint, slip-resistant (friction) or bearing, to be provided (see Clause 23).

4.1.4 If required, camber of beams, girders, and trusses shall be called for on the design drawings.

4.2 **Shop Details**

Shop details giving complete information necessary for the fabrication of the various members and components of the structure, including the required material and product standards and the location, type and size of all mechanical fasteners and welds, shall be prepared in advance of fabrication, and submitted for approval when so specified. Shop details shall distinguish clearly between mechanical fasteners and welds required for shop fabrication and those required in the field.

4.3 **Erection Diagrams**

Erection diagrams shall show the principal dimensions of the structure, piece marks and sizes of the members where necessary for approval, elevation of the column bases, all necessary dimensions and details fcr setting anchor bolts and all other information necessary for the assembly of the structure. (See also Clause 20.1.3.)

5. Material: Standards and Identification

5.1 Standards

5.1.1 General

Acceptable material and product standards and specifications (latest editions) for use under this Standard are listed in Clauses 5.1.2 to 5.1.8 inclusive. Materials and products other than those listed may also be used if approved. Approval shall be based on published specifications which establish the properties, characteristics, and suitability of the material or product to the extent and in the manner of those standards which are listed.

5.1.2 Structural Steel

CSA G40.21-M,
Structural Quality Steels.

5.1.3 Sheet Steel

ASTM A570,
Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality.

Other standards for structural sheet are listed in Clause 14. Only structural quality sheet standards which specify chemical composition and mechanical properties will be acceptable for use in the other clauses of this Standard. Mill test certificates which list the chemical composition and the mechanical properties shall be available, upon request, in accordance with Clause 5.2.1(a).

5.1.4 Cast Steel

CSA G28,
Carbon-Steel Castings for General Application;

ASTM A27,
Mild- to Medium-Strength Carbon-Steel Castings for General Application;

ASTM A148,
High-Strength Steel Castings for Structural Purposes;

ASTM A486,
Steel Castings for Highway Bridges.

5.1.5 Forged Steel

CSA G38,
Heavy Steel Shaft Forgings;

ASTM A521,
Steel, Closed-Impression Die Forgings for General Industrial Use;

ASTM A668,
Steel Forgings, Carbon and Alloy, for General

Industrial Use.

5.1.6 **Bolts**

ASTM A307,
Carbon Steel Externally and Internally Threaded Standard Fasteners;

ASTM A325,
High-Strength Bolts for Structural Steel Joints;

A325M,
High-Strength Bolts for Structural Steel Joints (Metric);

A490,
Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength

A490M,
High-Strength Steel Bolts, Classes 10.9 and 10.9.3 for Structural Steel Joints (Metric).

Note: Before specifying metric bolts, the designer should check on their current availability in the quantities required.

5.1.7 **Welding Electrodes**
CSA W48.1
Mild Steel Covered Arc-Welding Electrodes;

CSA W48.3,
Low-Alloy Steel Arc-Welding Electrodes;

CSA W48.4,
Solid Mild Steel Electrodes for Gas Metal-Arc Welding;

CSA W48.5,
Mild Steel Electrodes for Flux Cored Arc Welding;

CSA W48.6,
Bare Mild Steel Electrodes and Fluxes for Submerged-Arc Welding.

5.1.8 **Studs**
ASTM A108,
Steel Bars, Carbon, Cold-Finished, Standard Quality, Grades 1015 and 1018.

5.2 **Identification**

5.2.1 **Methods**
The materials and products used shall be identified

as to specification, including type or grade, if applicable, by one of the following means, except as provided in Clauses 5.2.2 and 5.2.3:

(a) Mill Test Certificates or Producer's Certificates satisfactorily correlated to the materials or products to which they pertain;

(b) Legible markings on the material or product made by its Producer in accordance with the applicable material or product standard.

5.2.2 **Unidentified Structural Steel**
Unidentified structural steel shall not be used, unless approved by the building designer. If the use of unidentified steel is authorized, F_y shall be taken as 210 MPa and F_u shall be taken as 380 MPa.

5.2.3 **Tests to Establish Identification**
Unidentified structural steel may be tested to establish identification when permitted by the building designer. Testing shall be done by an approved testing agency in accordance with CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel. The test results, taking into account both mechanical properties and chemical composition, shall form the basis for classifying the steel as to specification.

5.2.4 **Affidavit**
The fabricator, if requested, shall provide an affidavit stating that the materials and products which he has used in fabrication conform to the applicable material or product standards called for by the design drawings or specifications.

6. Design Requirements

6.1 General

As set out in this Standard, steel structures for buildings shall be designed to be serviceable during the useful life of the structure. The requirements related to serviceability such as camber, provisions for expansion and contraction, deflections and dynamic effects are examined at the specified load level. Other design requirements relate to corrosion protection and durability. The safety of the structure is achieved by limiting the stresses in members due to the specified loads to allowable values determined as a fraction of the limiting stress based on such conditions as yielding, fatigue, buckling, or fracture.

6.2 Requirements Under Specified Loads

6.2.1 **Deflections**

6.2.1.1 Steel members and frames shall be proportioned so that deflections are within acceptable limits for the nature of the materials to be supported and the intended use and occupancy.

6.2.1.2 In the absence of a more detailed evaluation see Appendix H for recommended values for deflections.

6.2.2 **Rain Loads**

6.2.2.1 Any roof which can accumulate water shall be designed for the load that can result from a 24 hour rainfall, or such greater load prescribed by the Regulatory Authority, on the horizontal projected area of the roof whether or not the surface is provided with drainage, such as rain water leaders. This applies particularly to large flat roofs in areas of heavy rainfall.

6.2.2.2 The distribution of rain load shall be determined taking into account the shape of the roof, including camber, with or without creep deflection due to dead load in the case of composite construction, and any loads likely to occur as a result of ponding.

6.2.2.3 The amplification of deflections due to ponding shall be considered. This amplification need not be considered provided that the roof stiffness meets the criterion:

(a) For one-way system of roof beams or decking simply supported on rigid supports

$$EI_b > 2\rho g S_b \left(\frac{L_b}{\pi}\right)^4$$

(b) For two-way system of roof joist on girders

$$\frac{2\rho g S_j}{EI_j}\left(\frac{L_j}{\pi}\right)^4 + \frac{2\rho g S_g}{EI_g}\left(\frac{L_g}{\pi}\right)^4 < 1$$

where

E = modulus of Elasticity
I = moment of inertia of the beam, joist, girder, or unit width of decking as applicable
L = span of the beam, joist or girder as applicable
S = spacing of the beam, joist, girder or unit width or decking as applicable
ρ = mass density of water, kg/m^3

6.2.3 **Camber**

6.2.3.1 Camber of beams, trusses or girders, if required, shall be called for on the design drawings. Generally trusses and crane girders of 25 000 mm or greater span should be cambered for approximately the dead-plus-half-live-load deflection (see also Clause 17 for requirements for open-web joists).

6.2.3.2 Any special camber requirements necessary to bring a loaded member into proper relation with the work of other trades shall be stipulated on the design drawings.

6.2.4 **Dynamic Effects**

6.2.4.1 Suitable provision shall be made in the design for the effect of live load which induces impact or vibration, or both. In severe cases, such as structural supports for heavy machinery which causes substantial impact or vibration when in operation, the possibility of harmonic resonance, fatigue, or unacceptable vibration shall be investigated.

6.2.4.2 Special consideration shall be given to floor systems susceptible to vibration, such as large open floor areas free of partitions, to ensure that such vibration is acceptable for the intended use and occupancy. (A guide on floor vibrations is contained in Appendix F of this Standard.)

6.2.4.3 Unusually flexible structures (generally those whose ratio of height to effectively resisting width exceeds 4:1) shall be investigated for lateral vibrations under dynamic wind load. Lateral accelerations of the structure shall be checked to ensure that such accelerations are acceptable for the intended use and occupancy. (Information on lateral accelerations under dynamic wind loads can be found in Appendix G.)

6.2.5 **Resistance to Fatigue**

Structural steelwork shall be designed to resist the effects of fatigue under the specified loads in accordance with Clause 15.

6.2.6 **Prevention of Permanent Deformation**

6.2.6.1 For composite beams unshored during construction, the stresses in the tension flange of the steel beam due to the loads applied before the concrete strength reaches $0.75f'_c$ plus the stresses at the same location due to the remaining specified loads considered to act on the composite section shall not exceed $0.90F_y$.

6.2.6.2 Slip-resistant (friction) joints, in which the design load is assumed to be transferred by the slip resistance of the clamped faying surfaces, shall be proportioned, using the provisions of Clause 13.12, to resist without slipping, the moments and forces induced by the specified loads (see Clause 23).

6.3 Requirements for Strength and Overturning

6.3.1 Strength

Structural steelwork shall be proportioned to resist moments and forces resulting from application of the loads acting in the most critical combination, taking into account the importance of the building, as specified in Clause 7.

6.3.2 Overturning

The building or structure shall be designed to resist overturning resulting from application of the loads acting in the most critical combination, taking into account the importance of the building as specified in Clause 7. See Clause 7.2.8.

6.3.3 Resistance to Earthquakes

6.3.3.1 In areas of known seismic activity buildings shall be designed to resist moderate earthquakes without significant damage and resist major earthquakes without collapse. Collapse is defined as the state which exists when exit of the occupants from the building has become impossible because of failure of the primary structure. The intent is to provide buildings with resistance to earthquake ground motion but not to slides, subsidence or active faulting in the immediate vicinity of the structure.

6.3.3.2 Unusual structures, highly irregular buildings and special-purpose industrial structures such as nuclear reactors, power plants and stacks should be treated as special problems with special design criteria in each instance, including possibly a dynamic analysis.

6.3.3.3 Structures designed to be resistant to earthquakes shall meet the requirements for structures analysed plastically as given in Clause 8.5 and in addition:

(a) the total drift per storey under the most severe design earthquake shall not exceed 0.03 of the storey height;

(b) beam webs yielding under cyclic loading, shall be designed so that local buckling does not occur.

6.3.3.4 In determining the lateral forces to be used in the design against earthquakes the type of construction, damping, ductility, and energy absorptive capacity of the structure shall be taken into consideration.*

*Information on a numerical coefficients reflecting this behaviour is available in the National Building Code of Canada 1985.

6.4 Other Requirements

6.4.1 Expansion and Contraction

Suitable provision shall be made for expansion and contraction commensurate with the service and erection conditions of the structure.

6.4.2 Corrosion Protection

6.4.2.1 Steelwork shall have sufficient corrosion protection to minimize any corrosion likely to occur in the service environment.

6.4.2.2 Corrosion protection shall be provided by means of suitable alloying elements in the steel, by protective coatings or by other effective means, either singly or in combination.

6.4.2.3 Localized corrosion likely to occur from entrapped water, excessive condensation, or from other factors shall be minimized by suitable design and detail. Where necessary, positive means of drainage shall be provided.

6.4.2.4 If the corrosion protection specified for steelwork exposed to the weather, or to other environments in which progressive corrosion can occur, is likely to require maintenance or renewal during the service life of the structure, the steelwork so protected (exclusive of fill plates and shims) shall have a minimum thickness of 4.5 mm.

6.4.2.5 The minimum required thickness of steelwork situated in a non-corrosive environment and therefore not requiring corrosion protection is governed by the provisions of Clause 11.

6.4.2.6 Interiors of buildings conditioned for human comfort may be generally assumed to be non-corrosive environments; however, the need for corrosion protection shall be assessed and protection shall be furnished in those where it is deemed to be necessary.

6.4.2.7 Corrosion protection of the inside surfaces of enclosed spaces permanently sealed from any external source of oxygen is unnecessary.

7. Loads and Safety Criterion

7.1 Specified Loads

7.1.1 Except as provided for in Clauses 7.1.2 and 7.1.3, the following specified loads, forces and effects and influences as specified by the Regulatory Authority shall be considered in the design of a building and its structural steelwork:

D - Dead loads, including the mass of steelwork and all permanent materials of construction, partitions and stationary equipment multiplied by the acceleration due to gravity to convert mass to force, and the forces due to prestressing;

L - Live loads, including loads due to intended use and occupancy of the building, movable equipment, rain, soil or hydrostatic pressure, impact, and any other live loads stipulated by the applicable building by-law or the Regulatory Authority;

Q - Wind or earthquake loads;

T - Influences resulting from temperature changes, shrinkage or creep of component materials, or from differential settlement.

The minimum specified value of these loads as established by the authority having jurisdiction shall be increased to account for dynamic effects where applicable.

7.1.2 Where a building or a structural member can be expected to be subjected to loads, forces or other effects not listed in Clause 7.1.1 such effects shall be taken into account in the design based on the most appropriate information available.

7.1.3 If it can be shown by engineering principles, or if it is known from experience, that neglect of some or all of the effects due to temperature changes, shrinkage or creep of componenet materials or from differential settlement does not affect the structural safety or serviceability, they need not be considered in the calculations.

7.1.4 Suitable provision shall be made for loads imposed on the steel structure during its erection. During subsequent construction of the building, suitable provision shall be made to support the construction loads on the steel structure with an adequate margin of safety.

7.2 Safety Criterion and Effect of Specified Loads

7.2.1 Safety Criterion

Structural steelwork shall be proportioned to sustain the specified loads and forces. When the working stress method of design is used, the allowable stresses given in this Standard shall not be exceeded. When the plastic method of design is used the load factors given in Clause 31.3 shall be employed.

7.2.2 Effect of specified loads shall be taken as the most unfavorable combination of:

$$D + \psi\gamma(L + Q + T)$$

where ψ is given in Clause 7.2.3 and γ is given in Clause 7.2.4.

7.2.3 The load combination factor, ψ, shall be taken as follows:

(a) When one of L, Q or T act, $\psi = 1.00$;

(b) When two of L, Q or T act, $\psi = 0.75$;

(c) When all of L, Q and T act, $\psi = 0.66$.

7.2.4 The importance factor, γ, shall be not less than 1.0 for all buildings, except that for buildings where it can be shown that collapse is not likely to cause injury or other serious consequences, it shall be not less than 0.8.

7.2.5 When loads other than D counteract D in a structural member or joint, special caution shall be exercised by the designer to ensure adequate safety for possible load reversal.

7.2.6 Tension and compression members and their connections, in which a load reversal would occur by application of four-thirds of the live load and full dead load shall be proportioned for the resultant load of opposite sign as well as for other load combinations as given in Clause 7.2.3.

7.2.7 Tension and Compression members and their connections, in which a load reversal would occur by application of four-thirds of the specified lateral loads acting in conjunction with dead load and no gravity live load shall be proportioned for the resultant load of opposite sign as well as for other combinations as given in Clause 7.2.3.

7.2.8 A building shall be proportioned to resist an overturning moment and sliding force of not less than twice that due to the loads acting on the structure when the structure is considered as an entire unit acting on or anchored to its bearing stratum or supporting structure. The resistance to overturning shall be calculated as the sum of the stabilizing moment of the dead load only, plus the ultimate resistance of any anchoring devices.

8. Analysis of Structure

8.1 General

8.1.1 In proportioning the structure to meet the various design requirements of Clause 6, the methods of analysis given in this Clause shall be used. The distribution of internal forces and bending moments shall be determined under the specified loads.

8.1.2 Two basic types of construction and associated design assumptions, designated "Continuous" and "Simple" are permitted for all or part of a structure under this Standard. The distribution of internal forces and bending moments throughout the structure will depend on the type or types of construction chosen and the forces to be resisted.

8.2 Continuous Construction

In continuous construction, the beams, girders and trusses are rigidly framed, or are continuous over supports. Connections are generally designed to resist the bending moments and internal forces computed by assuming that the original angles between intersecting members remain unchanged as the structure is loaded.

8.3 Simple Construction

8.3.1 Simple construction assumes that the ends of beams, girders and trusses are free to rotate under load in the plane of loading. Resistance to lateral loads, including sway effects, shall be ensured by a suitable system of bracing or shear walls or by the design of part of the structure as continuous construction, except as provided in Clause 8.3.2.

8.3.2 A building frame designed to support gravity loads on the basis of simple construction may be proportioned to resist lateral loads due to wind or earthquake by distributing the moments resulting from such loading among selected joints of a frame by a recognized empirical method provided that:

(a) The connection and connected members are proportioned to resist the moments and forces caused by lateral loads;

(b) The connected members have solid webs;

(c) The beam or girder can support the full gravity load when assumed to act as a simple beam;

(d) The connection has adequate capacity for inelastic rotation when subjected to full gravity and lateral loads;

(e) The stresses in the mechanical fasteners or welds of the connection do not exceed the values given in Clause 13 when the connection is assumed to be carrying the moment at which inelastic rotation would occur.

(f) In assessing the stability of the structure, the flexibility of the connection is considered.

8.4 **Elastic Analysis**

Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by an analysis which assumes that individual members behave elastically. The computed stresses shall not exceed the allowable stresses given in Clauses 13 and, if applicable, 15.

8.5 **Plastic Analysis**

Under a particular loading combination, the forces and moments throughout all or part of the structure produced by the specified loads multiplied by the prescribed load factors may be determined by a plastic analysis provided that:

(a) The steel used has $F_y < 0.80F_u$ and exhibits the load-strain characteristics necessary to achieve moment redistribution;

(b) The width-thickness ratios meet the requirements of Class 1 sections as given in Clause 11.2;

(c) The members are braced laterally in accordance with the requirements of Clause 13.7;

(d) Web stiffeners are supplied on a member at a point of load application or where a plastic hinge would form;

(e) Splices in beams or columns are designed to transmit 1.1 times the maximum computed moment under factored loads at the splice location or $0.25M_p$, whichever is greater.

(f) Members are not subject to repeated heavy impact or fatigue;

(g) The influence of inelastic deformation on the strength of the structure shall be taken into account. (See also Clause 8.6.)

8.6 Stability Effects

8.6.1 The analyses referred to in Clauses 8.4 and 8.5 shall include the sway effects produced by the vertical loads acting on the structure in its displaced configuration, unless the structure is designed in accordance with the provisions of Clause 8.6.3.

For certain types of structures where the vertical loads are small, where the structure is relatively stiff and where the lateral load resisting elements are well distributed, the sway effects may not have a significant influence on the design of the structure (see Clause 9.3.2(b)).

8.6.2 For structures in which the sway effects have been included in the analysis to determine the design moments and forces (see Appendix I) the effective length factors for members shall be based on the side-sway prevented condition (see Clause 9.3.2(a)) and,

(a) Where a loading combination produces significant relative lateral displacements of the column ends, the sway effects produced by the vertical loads acting on the displaced structure shall be multiplied by 1.7 to account for non-linearity of the behaviour of the structure;

(b) However, in no case shall the sway effects be taken as less than those calculated by assuming that 1.7 times the vertical loads act on the structure assumed to be displaced by an amount equal to the maximum out-of-plumbness consistent with the erection tolerances specified in Clause 29.7.1;

(c) A deflected configuration in which the erection tolerances are opposite in sense in adjacent storeys, may produce sway effects which govern the design of beam-to-column connections, diaphragms and other elements.

8.6.3 For structures in which the sway effects have not been included in the analysis, the use of effective length factors greater than 1.0 (side-sway permitted case) for the design of columns, provides an approximate method of accounting for the sway effects in moment resisting frames (see Clause 9.3.3). This provision shall not be used for structures analysed in accordance with Clause 8.5.

8.6.4 For structures analysed plastically in accordance with Clause 8.5 in which the sway effects have been included in the analysis to determine the design moments and forces, the provisions of Clause 8.6.2 shall apply except that the sway effects shall be those produced by the factored loads rather than 1.7 times those produced by specified loads.

9. Design Lengths of Members

9.1 Simple Span Flexural Members

Beams, girders, and trusses may be designed on the basis of simple spans whose length may be taken as the distance between centres of gravity of supporting members. Alternatively, the span length of beams and girders may be taken as the actual length of such members measured centre-to-centre of end connections. The length of trusses designed as simple spans may be taken as the distance between the extreme working points of the system of triangulation employed. In all cases the design of columns or other supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder, or truss may actually be connected or supported.

9.2 Continuous Span Flexural Members

Beams, girders, or trusses having full or partial end restraint due to continuity or cantilever action, shall be proportioned to carry all moments, shears, and other forces at any section assuming the span, in general, to be the distance between centres of gravity of supporting members. Supporting members shall be proportioned to carry all moments, shears, and other forces induced by the continuity of the supported beam, girder, or truss.

9.3 Compression Members

9.3.1 Compression members shall be designed on the basis of their effective length (KL), the product of effective length factor (K), and unbraced length (L). Unless otherwise specified in this Standard the unbraced length (L) shall be taken as the length of the compression member centre-to-centre of restraining members. The unbraced length may differ for different cross-sectional axes of the compression members. At the bottom storey of a multistorey structure, or for a single-storey structure, (L) shall be taken as the length from the top of the base plate to the centre of restraining members at the next higher level.

9.3.2 The effective length factor (K) shall be taken as 1.0 for compression members of frames:

(a) In which sway effects (see Clause 8.6.2) have been included in the analysis used to determine the design moments and forces; or

(b) In which the sway effects in addition to the lateral loads are resisted by bracing or shear walls,

unless the degree of rotational restraint afforded at the ends of the unbraced lengths shows that a value of K less than 1.0 is applicable. (For recommended values of K and a method of computing K, based on rotational restraint, see Appendices B and C respectively, side-sway prevented case.)

9.3.3 For structures with moment resisting frames in which sway effects have not been included in the analysis used to determine the design moments and forces (see Clause 8.6.3), the effective length factor shall be determined from the degree of rotational and translational restraint afforded at the ends of the unbraced length, but shall be not less than 1. (For recommended values of K and a method of computing K, see Appendices B and C respectively, side-sway permitted case.)

9.3.4 **Compression Members in Trusses**

Unless otherwise specified in this Standard or unless analysis shows that a smaller value is applicable, the effective length factor (K) shall be taken as 1.0 for compression members in trusses.

10. <u>Slenderness Ratios</u>

10.1 **General**

The slenderness ratio of a compression member shall be taken as the ratio of effective length (KL) to the corresponding radius of gyration (r). The slenderness ratio of a tension member shall be taken as the ratio of unbraced length (L) to the corresponding radius of gyration.

10.2 **Maximum Slenderness Ratio**

10.2.1 The slenderness ratio of a compression member shall not exceed 200.

10.2.2 The slenderness ratio of a tension member shall not exceed 300. This limit may be waived if other means are provided to control flexibility, sag, vibration, and slack in a manner commensurate with the service conditions of the structure, or if it can be shown that such factors are not detrimental to the performance of the structure or of the assembly of which the member is a part.

11. Width-Thickness Ratios: Compression Elements

11.1 **Classification of Sections**

11.1.1 For the purposes of this Standard, structural sections shall be designated as Class 1, 2, 3 or 4 depending on the maximum width-thickness ratios of their elements subject to compression, and as otherwise specified in Clause 11.1.3.

11.1.2 **Class 1** sections (plastic design sections) will permit attainment of the plastic moment and subsequent redistribution of bending moment.

Class 2 sections (compact sections) will permit attainment of the plastic moment but need not allow for subsequent moment redistribution.

Class 3 sections (non-compact sections) will permit attainment of the yield moment.

Class 4 sections will generally have local buckling of elements in compression as the limit state of structural capacity.

11.1.3 Class 1 sections shall, when subject to flexure, have an axis of symmetry in the plane of loading and shall, when subject to axial compression, be doubly symmetric. Class 2 sections shall, when subject to flexure, have an axis of symmetry in the plane of loading unless the effects of asymmetry of the section are included in the analysis.

11.2 **Width and Thickness**

11.2.1 For elements supported along one edge only, parallel to the direction of compressive force, the width shall be taken as follows:

(a) For plates, the width (b) is the distance from the free edge to the first row of fasteners or line of welds;

(b) For legs of angles, flanges of channels and zees, and stems of tees, the width (b) is the full nominal dimension;

(c) For flanges of beams and tees, the width (b) is one-half the full nominal dimension.

11.2.2 For elements supported along two edges parallel to the direction of compressive force the width shall be taken as follows:

(a) For flange or diaphragm plates in built-up sections the width (b) is the distance between adjacent lines of fasteners or lines of welds;

(b) For flanges of rectangular hollow structural sections the width (b) is the clear distance between webs less the inside corner radius on each side;

(c) For webs of built-up sections the width (h) is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used;

(d) For webs of hot rolled sections the width (h) is the clear distance between flanges.

11.2.3 The thickness of elements is the nominal thickness. For tapered flanges of rolled sections, the thickness is the nominal thickness halfway between a free edge and the corresponding face of the web.

11.3 Maximum Width-Thickness Ratios of Elements Subject to Compression

The width-thickness ratio of elements subject to compression shall not exceed the limits given in Table 1 for the specified section classification.

12. Gross and Net Areas

12.1 Application

In general, tension members shall be proportioned on the basis of net area and compression members on the basis of gross area. (For beams and girders see Clause 15.)

Table 1
Width-Thickness Ratios: Compression Elements

	Section Classification			
Description of Element	**Class 1 Plastic Design**	**Class 2 Compact**	**Class 3 Non-compact**	**Class 4* Slender**
Legs of angles and elements supported along one edge, except as noted	--	--	$\frac{b}{t} \leqslant \frac{200}{\sqrt{F_y}}$	See Clause 13
Angles in continuous contact with other elements; plate girder stiffeners	--	--	$\frac{b}{t} \leqslant \frac{200}{\sqrt{F_y}}$	See Clause 13
Stems of T sections	$\frac{b}{t} \leqslant \frac{145\dagger}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{170\dagger}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{340}{\sqrt{F_y}}$	See Clause 13
Flanges of I or T sections; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact‡	$\frac{b}{t} \leqslant \frac{145}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{170}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{200}{\sqrt{F_y}}$	See Clause 13
Flanges of channels	--	--	$\frac{b}{t} \leqslant \frac{200}{\sqrt{F_y}}$	See Clause 13
Flanges of rectangular hollow structural sections	$\frac{b}{t} \leqslant \frac{420}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{525}{\sqrt{F_y}}$	$\frac{b}{t} \leqslant \frac{670}{\sqrt{F_y}}$	See Clause 13

Table 1 continued

Description of Element	Section Classification: Class 1 Plastic Design	Class 2 Compact	Class 3 Non-compact	Class 4* Slender
Flanges of box sections, flange cover plates and diaphragm plates, between lines of fasteners or welds	$\frac{b}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b}{t} \leq \frac{670}{\sqrt{F_y}}$	See Clause 13
Perforated cover plates	--	--	$\frac{b}{t} \leq \frac{840}{\sqrt{F_y}}$	--
Webs in axial compression	$\frac{h}{w} \leq \frac{670}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{670}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{670}{\sqrt{F_y}}$	See Clause 13
Webs in flexural compression	$\frac{h}{w} \leq \frac{1100}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{1370}{\sqrt{F_y}}$	$\frac{h}{w} \leq \frac{1810}{\sqrt{F_y}}$	See Clause 13
Webs in combined flexural and axial compression	$\frac{h}{w} \leq \frac{1100}{\sqrt{F_y}}\left(1 - 1.40\frac{f_a}{F'_a}\right)$ §	when $\frac{f_a}{F'_a} \leq 0.15$, $\frac{h}{w} \leq \frac{1370}{\sqrt{F_y}}\left(1 - 1.28\frac{f_a}{F'_a}\right)$ when $\frac{f_a}{F'_a} > 0.15$, $\frac{h}{w} \leq \frac{1180}{\sqrt{F_y}}\left(1 - 0.43\frac{f_a}{F'_a}\right)$	when $\frac{f_a}{F'_a} \leq 0.15$, $\frac{h}{w} \leq \frac{1810}{\sqrt{F_y}}\left(1 - 1.69\frac{f_a}{F'_a}\right)$ when $\frac{f_a}{F'_a} > 0.15$, $\frac{h}{w} \leq \frac{1470}{\sqrt{F_y}}\left(1 - 0.54\frac{f_a}{F'_a}\right)$	See Clause 13
Circular hollow sections	$\frac{D}{t} \leq \frac{13\ 000}{F_y}$	$\frac{D}{t} \leq \frac{18\ 000}{F_y}$	$\frac{D}{t} \leq \frac{23\ 000}{F_y}$	--

* Class 4 includes all sections not otherwise specified.

† See Clause 11.1.3

‡ Can be considered as Class 1 or Class 2 sections if angles are continuously connected by adequate mechanical fasteners or welds and there is an axis of symmetry in the plane of loading.

§ $\frac{h}{w}$ need not be less than $\frac{670}{\sqrt{F_y}}$

12.2 Gross Area

Gross area shall be computed by summing the products of the thickness and gross width of each element, as measured normal to the axis of the member.

12.3 Net Area

12.3.1 Net area shall be computed by summing the products of the thickness and the net width of each element, as measured normal to the axis of the member. Net width and area of parts containing holes shall be computed in accordance with Clause 12.3.3.

12.3.2 **Dimensions of Holes**

In computing net area the width of the bolt holes normal to the axis of the member shall be assumed 2 mm larger than the hole dimension specified.

12.3.3 For a series of holes extending across a part of any diagonal or zigzag line, the net width of the part shall be computed by deducting from the gross width the sum of all hole widths in the series and adding for each gauge distance (g) in the series the quantity:

$$\frac{s^2}{4g}$$

where

s = longitudinal spacing (pitch) in millimetres of any two successive holes

g = transverse spacing (gauge) in millimetres of the same two holes

12.3.4 The critical net area of the part is obtained from that series of holes which gives the least net width; however, the net area through one or more holes shall not be taken as greater than the following limits:

(a) $0.85A_g$ when $F_y/F_u \leq 0.75$

(b) $0.90A_g$ when $0.75 < F_y/F_u \leq 0.85$

(c) $0.95A_g$ when $0.85 < F_y/F_u$

12.3.5 For angles the gross width shall be the sum of the widths of the legs minus the thickness. The gauge for holes in opposite legs shall be the sum of the gauges from the heel of the angle minus the thickness.

12.3.6 In computing the net area across plug or slot welds the weld metal shall not be taken as adding to the net area.

12.4 Pin-Connected Tension Members

12.4.1 In pin-connected tension members, the net area across the pin hole, normal to the axis of the member, shall be at least 1.33 times the cross-sectional area of the body of the member. The net area of any section on either side of the axis of the member measured at an angle of 45° or less to the axis of the member, shall be not less than 0.9 times the cross-sectional area of the body of the member.

12.4.2 The distance from the edge of the pin hole to the edge of the member, measured transverse to the axis of the member, shall not exceed 4 times the thickness of the material at the pin hole.

12.4.3 The diameter of a pin hole shall be not more than 1 mm larger than the diameter of the pin.

13. Allowable Stresses - Members and Connections

13.1 General

13.1.1 Where applicable, the provisions of Clauses 8, 9-11, 15-18 shall supplement the provisions of Clause 13.

13.1.2 Allowable stresses are given in MPa unless otherwise stated.

13.2 Axial Tension

The allowable tensile stress, F_t, developed by a member subjected to an axial tension force shall be taken as:

(a) The lesser of

(i) $F_t = 0.60F_y$ when $A_n/A_g > F_y/F_u$

$= 0.60\left(F_u \dfrac{A_n}{A_g}\right)$ when $A_n/A_g < F_y/F_u$

(ii) $F_t = 0.50F_u$

(b) On net area at pin connections

$$F_t = 0.45F_y$$

(c) On unthreaded body area of threaded parts

$$F_t = 0.40F_y$$

(If threads are upset, $F_t = 0.60F_y$ on the critical net area.)

13.3 Axial Compression

13.3.1 The allowable axial compressive stress for W shapes and for hollow structural sections manufactured according to CSA Standard CAN3-G40.20-M, Class C (cold formed non-stress relieved) and conforming to the requirements of Clause 11 of this Standard for Class 1, 2 or 3 sections, shall be taken as*:

(a) $0 \leq \lambda \leq 0.15$, $F_a = 0.60F_y$

(b) $0.15 < \lambda \leq 1.0$, $F_a = 0.60F_y(1.035 - 0.202\,\lambda - 0.222\,\lambda^2)$

(c) $1.0 < \lambda \leq 2.0$, $F_a = 0.60F_y(-0.111 + 0.636\,\lambda^{-1} + 0.087\,\lambda^{-2})$

(d) $2.0 < \lambda \leq 3.6$, $F_a = 0.60F_y(0.009 + 0.877\,\lambda^{-2})$

(e) $3.6 < \lambda$, $F_a = 0.60F_y\,\lambda^{-2} = \left[\dfrac{1\ 180\ 000}{(KL/r)^2}\right]$

where $\lambda = \dfrac{KL}{r}\sqrt{\dfrac{F_y}{\pi^2 E}}$

Values of $\sqrt{\dfrac{F_y}{\pi^2 E}}$ and $\dfrac{F_y}{\pi^2 E}$ to compute λ and λ^2 respectively are given in Table 2.

* These expressions defining F_a are based on column strength predictions for W shapes up to 610 mm deep and can be assumed to be valid for other doubly symmetric Class 1, 2 or 3 sections, except for solid round non-stress relieved cold straightened bars greater than 50 mm diameter (refer to CSA Standard S37-M, Antenna Towers and Antenna Supporting Structures). Welded H-shapes should have flange edges flame cut. Singly symmetric, asymmetric or cruciform sections should be checked as to whether torsional-flexural buckling is critical. Curves similar to those described by the equations in Clause 13.3.1 but taking into account differences in residual stress patterns, shapes, etc., for fully stress relieved sections, hollow structural sections and jumbo sections are given in "Guide to Stability Design Criteria for Metal Compression Members", 3rd Edition, published by the Structural Stability Research Council.

Table 2

Values of $\sqrt{\frac{F_y}{\pi^2 E}}$ and $\frac{F_y}{\pi^2 E}$

F_y	$\sqrt{\frac{F_y}{\pi^2 E}}$	$\frac{F_y}{\pi^2 E}$
230	0.0108	0.000 116
260	0.0115	0.000 132
300	0.0123	0.000 152
350	0.0133	0.000 177
380	0.0139	0.000 192
400	0.0142	0.000 203
480	0.0156	0.000 243
700	0.0188	0.000 355

13.3.2 The allowable axial compressive stress for hollow structural sections manufactured according to CSA Standard CAN3-G40.20-M, Class H (hot formed or cold formed stress relieved) and conforming to the requirements of Clause 11 of this Standard for Class 1, 2 or 3 sections, shall be taken as:

(a) $0 < \lambda < 0.15$, $F_a = 0.60F_y$

(b) $0.15 \leq \lambda < 1.2$, $F_a = 0.60(0.990 + 0.122\lambda - 0.367\lambda^2)$

(c) $1.2 < \lambda < 1.8$, $F_a = 0.60F_y(0.051 + 0.801\lambda^{-2})$

(d) $1.8 < \lambda < 2.8$, $F_a = 0.60F_y(0.008 + 0.942\lambda^{-2})$

(e) $2.8 < \lambda$, $F_a = 0.60F_y\lambda^{-2} = \left[\frac{1\,180\,000}{(KL/r)^2}\right]$

where $\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}$

Values of $\sqrt{\frac{F_y}{\pi^2 E}}$ and $\frac{F_y}{\pi^2 E}$ to compute λ and λ^2 respectively are given in Table 2.

13.3.3 The allowable axial compressive stress, F_a, developed by a member subject to an axial compressive force and designated as a Class 4 section according to Clause 11 shall be determined as follows:

(a) For sections less or equal to 4.5 mm in thickness and which are not hollow structural sections, the allowable axial compressive stress shall be calculated in accordance with Clause 14.

(b) For hollow structural sections and sections greater than 4.5 mm in thickness which can be shown not to be critical in torsional buckling or not subject to torsional flexural buckling, the allowable axial compressive stress shall be calculated in accordance with the requirements of 13.3.1 or 13.3.2, as applicable. The area, A, shall be taken as the effective area determined in accordance with Clause 14. The slenderness ratio shall be calculated using gross section properties.

(c) Singly symmetric sections which may be subject to torsional flexural buckling and channels with unstiffened flanges shall be designed by rational analysis.

13.3.4 The allowable axial compressive stress, F_a on the gross area of a web stiffener shall be taken as:

$$F_a = 0.60F_y$$

13.4 Shear

13.4.1 Elastic Analysis

Except as noted in Clause 13.4.2, the allowable shear stress, F_y developed by the web of a flexural member subjected to shear shall be taken as follows:

(a) $\frac{h}{w} \leq 439\sqrt{\frac{k_v}{F_y}}$ .... $F_v = 0.40F_y$

(b) $439\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 502\sqrt{\frac{k_v}{F_y}}$ .... $F_v = \frac{175\sqrt{F_y k_v}}{(h/w)}$

(c) $502\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 621\sqrt{\frac{k_v}{F_y}}$ ..... $F_v = \left(\frac{175\sqrt{F_y k_v}}{(h/w)}\right)\tau + \eta F_y$

(d) $621\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq \frac{83\ 000}{F_y}$ ... $F_v = \left(\frac{109\ 000 k_v}{(h/w)^2}\right)\tau + \eta F_y$

where

F_v = allowable shear stress (MPa)

k_v = shear buckling coefficient

$$= 4 + \frac{5.34}{(a/h)^2} \text{ when } a/h < 1$$

$$= 5.34 + \frac{4}{(a/h)^2} \text{ when } a/h > 1$$

$$\tau = 1 - \frac{0.866}{\sqrt{1 + (a/h)^2}}$$

$$\eta = \frac{0.50}{\sqrt{1 + (a/h)^2}}$$

a/h = aspect ratio, ratio of distance between stiffeners to web depth

For unstiffened webs, $a/h = \infty$ and $\tau = 1$, $\eta = 0$, $k_v = 5.34$.

The values given in Table 3 may be used. The gross area of a web shall be taken as the product of the web depth (h) and the web thickness (w), except that for rolled shapes the overall depth (d) may be substituted for h.

Table 3
Coefficients for Shear Formulae

a/h	τ	η	k_v
0.25	0.160	0.485	89.4
0.33	0.178	0.475	52.2
0.50	0.225	0.447	25.4
0.67	0.280	0.415	16.0
0.75	0.307	0.400	13.5
1.00	0.388	0.354	9.34
1.25	0.459	0.312	7.90
1.50	0.520	0.277	7.12
1.75	0.570	0.248	6.65
2.00	0.613	0.224	6.34
2.25	0.648	0.203	6.13
2.50	0.678	0.186	5.98
2.75	0.704	0.171	5.87
3.00	0.726	0.158	5.78
Infinity	1.000	0	5.34

13.4.2 **Plastic Analysis**

Unless reinforced by diagonal stiffeners or a doubler plate, the webs of columns, beams, and girders shall be proportioned so that

$V_f < 0.55F_y wd$

where

V_f = shear force from factored loads
w = web thickness
d = depth of member

Note: This clause is to be used only in conjunction with Clause 31.

13.4.3 **Maximum Slenderness**

The slenderness ratio (h/w) of a web shall not exceed:

$83\ 000/F_y$

where

F_y = specified minimum yield point of the compression flange steel (see Clause 16.3).

This limit may be waived if analysis indicates that compression flange buckling into the web will not occur at 1.67 times the flange design stress.

13.4.4 **Gusset Plates**

The average shear stress of the gross area of gusset plates shall be taken as:

$F_v = 0.30F_y$

13.4.5 **Pins**

The average shear stress on nominal area of pins shall be taken as:

$F_v = 0.40F_y$

13.4.6 **Combined Shear and Moment in Girders**

Transversely stiffened girders depending on tension field action to carry shear, with $h/w > 502\sqrt{k_v/F_y}$ shall be proportioned in such a way that the following limits are observed.

$$\frac{f_v}{F_v} < 1.0$$

$$\frac{f_b}{F_b} \leq 1.0$$

$$0.727 \frac{f_b}{F_b} + \frac{0.455 f_v}{F_v} \leq 1$$

where

F_v is established according to Clause 13.4.
F_b is established according to Clause 13.5.

13.5 Bending

The allowable bending stress, F_b, developed by a member subjected to bending moments shall be as given in this Clause.

13.5.1 I-shaped Members and Channels Prevented from Twisting

(a) Tension

$F_{bt} = 0.66F_y$ for Class 1 and 2 sections

$F_{bt} = 0.60F_y$ for Class 3 and 4 sections

(b) Compression; Class 1, 2 and 3 sections

(i) When $F_1 > \frac{2}{3} F_{bt}$

$$F_{bc} = 1.15F_{bt}\left(1 - \frac{0.28F_{bt}}{F_1}\right)$$

but not greater than F_{bt}

(ii) When $F_1 \leq \frac{2}{3} F_{bt}$

$$F_{bc} = F_1$$

where

$$F_1 = \sqrt{(F_2)^2 + (F_3)^2}$$

$$F_2 = \frac{83\ 800}{Ld/A_f}$$

$$F_3 = \frac{1\ 018\ 000}{(L/r_t)^2} \text{ for I-shaped members}$$

$$= 0 \qquad \text{for channels}$$

where

L = unsupported length of compression flange in millimeters

(c) Compression; Class 4 sections

(i) When both the web and the compression flange fall within Class 4 of Table 1, the allowable stress shall be determined in accordance with Clause 14. The calculated value, F_y', applicable to cold formed members, shall be determined by using only the values for F_y and F_u that are specified in the relevant structural steel material standard;

(ii) For beams or girders whose flanges meet the requirements of Class 3 and whose webs exceed the limits for Class 3, see Clause 16;

(iii) For beams or girders whose webs meet the requirements of Class 3 and whose flanges exceed the limits for Class 3 the critical stress corresponding to local buckling of compression elements, F_{cr}, shall be determined in accordance with Clause 14. Alternatively, the yield stress, F_y, may be used with an effective section modulus, S_e,

where

S_e = the effective section modulus determined using an effective flange width of $670t/\sqrt{F_y}$ for flanges supported along two edges parallel to the direction of stress and an effective width of $200t/\sqrt{F_y}$ for flanges supported along one edge parallel to the direction of stress. In no case, for flanges supported along one edge, shall b/t exceed 60.

13.5.2 Hollow Structural Sections

Tension and Compression
$F_b = 0.66F_y$ for Class 1 and 2 sections
$F_b = 0.60F_y$ for Class 3 sections

For hollow structural sections the laterally unsupported length of the portion of a rectangular section in compression due to bending shall not exceed 17 200/F_y times its width unless analysis would show that a greater unsupported length may be used.

13.5.3 Tee Sections with Axis of Symmetry in Plane of Bending

Tension and Compression

$F_b = 0.66F_y$ for Class 1 and 2 sections

$F_b = 0.60F_y$ for Class 3 sections

For the tee sections the laterally unsupported length of the portion of the tee in compression due to bending shall not exceed $630r_y/\sqrt{F_y}$ where r_y is the radius of gyration of the tee about the axis of symmetry, unless analysis would show that a greater unsupported length may be used.

13.5.4 Sections without an Axis of Symmetry in Plane of Bending (Except Channels)

Tension and compression when the section is prevented from lateral buckling and twisting

$F_b = 0.60F_y$

13.5.5 Solid Sections

(a) Rounds and pins
$F_b = 0.90F_y$ for tension and compression

(b) Rectangles and bearing plates
$F_b = 0.75F_y$ for tension and compression

13.5.6 Continuous and Fixed-ended Beams Supporting Gravity Loads: Plastic Design Sections

Continuous or fixed-ended beams, exclusive of cantilevers, meeting the width-thickness limitations specified in Clause 31 for plastic design sections and not subject to web shear stress (f_v) in excess of $0.33F_y$, may be proportioned to support gravity loads on the following basis:

(a) Bending moments shall be determined by elastic theory;

(b) Within any span the positive bending stress computed on the assumption that the span is fully loaded and simply supported shall not exceed $1.33F_y$;

(c) The maximum bending stress considering the continuity of the structure shall not exceed $0.75F_y$;

(d) The distance from the face of the support to the point of contraflexure shall not exceed

$980r_y/\sqrt{F_y}$ when the flange in compression is unbraced within this distance, nor $1535r_y/\sqrt{F_y}$

when that flange is braced at a point not more than $550r_y/\sqrt{F_y}$ from the face of the support;

(e) The flange in compression between points of contraflexure, in the region of positive bending shall be braced at intervals not exceeding $550r_y/\sqrt{F_y}$.

If the negative moment of the beam is resisted by a column rigidly framed to that beam, the allowable bending stress in the column resulting from the assumption that the beam-to-column connection provides complete fixity under gravity load bending moment shall not exceed at the connection:

(i) $0.75F_y$ if the column meets the requirements of a Class 1 section.

(ii) $0.66F_y$ if the column meets the requirements of a Class 2 section.

(iii) $0.60F_y$ if the column meets the requirements of a Class 3 section.

13.6 **Lateral Bracing for Members in Structures Analysed Plastically**

Members in structures or portions of structures in which the distributions of moments and forces have been determined by a plastic analysis shall be braced to resist lateral and torsional displacement at all hinge locations. The laterally unsupported distance, L_{cr}, from such braced hinge locations to the nearest adjacent point on the frame similarly braced shall not exceed:

$$L_{cr} = 550r_y/\sqrt{F_y} \text{ for } \frac{M_f}{M_p} > 0.5$$

$$L_{cr} = 980r_y/\sqrt{F_y} \text{ for } \frac{M_f}{M_p} < 0.5$$

where

$\frac{M_f}{M_p}$ is equal to the ratio of the smaller moment to the larger moment at opposite ends of the unbraced length, in the plane of bending considered; positive when the member is bent in single curvature and negative when bent in double curvature and

$$-1.0 < \frac{M_f}{M_p} < 1.0$$

Both bracing requirements should be checked and the more severe shall govern the location of the braced point. Bracing is not required at the location of the last hinge to form in the failure mechanism assumed as the basis for proportioning the structure. Except for the regions specified above, the maximum unsupported length of members in structures analysed plastically need be not less than that which would be permitted for the same members in structures analysed elastically.

13.7 Axial Compression and Bending

13.7.1 Member Strength and Stability

Members required to resist bending moments and an axial compressive force shall be proportioned so that:

(a) $$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

(b) $$\frac{f_a}{F_a} + \frac{\omega_x f_{bx}}{F_{bx}\left(1 - \frac{f_a}{F'_{ex}}\right)} + \frac{\omega_y f_{by}}{F_{by}\left(1 - \frac{f_a}{F'_{ey}}\right)} \leq 1.0$$

where

F_a is defined in Clause 13.3

F_{bx} is defined in Clause 13.5

$$F'_e = \frac{1\ 180\ 000}{\left(\frac{KL}{r}\right)^2}$$

ω_x, ω_y are defined in Clause 13.7.2.

When $f_a/F_a \leq 0.15$ the value of each of $(1 - f_a/F'_{ex})$, $(1 - f_a/F'_{ey})$, ω_x, and ω_y may be assumed equal to 1.

13.7.2 Values of ω

Unless otherwise determined by analysis, the following values shall be used for ω:

(a) Members not subjected to transverse loads between supports:

(i) For members of frames analysed in accordance with Clause 8.6.2,
$\omega = 0.6 + 0.4M_1/M_2$ for members bent in single curvature
$\omega = 0.6 - 0.4M_1/M_2$ for members bent in double curvature, but not less than 0.4

where
M_1/M_2 = ratio of the smaller moment to the larger moment at opposite ends of the unbraced length, in the plane of bending considered;

(ii) For members of frames analysed in accordance with Clause 8.6.3,
ω = 0.85 for members bent in double curvature or subject to moment at one end
ω = 1.0 for members bent in single curvature due to moments at both ends

(b) Members subjected to distributed load or series of point loads between supports;

ω = 1.0

(c) Members subjected to a concentrated load or moment between supports: ω = 0.85.

For the purpose of design, members subjected to concentrated load (or moment) between supports (e.g., crane columns) may be considered to be divided into two segments at the point of load (or moment) application. Each segment shall then be treated as a member which depends on its own flexural stiffness to prevent side-sway in the plane of bending considered and ω shall be taken as 0.85. In computing the slenderness ratio KL/r, for use in Clause 13.7, the total length of the member shall be used.

13.8 Axial Tension and Bending

Members required to resist both bending moments and an axial tensile force shall be proportioned so that:

(a) $\frac{f_t}{F_t} + \frac{f_{bt}}{F_{bt}} < 1.0$

where

F_{bt} = tensile bending stress permitted if bending alone was present

F_t = axial tensile stress permitted if axial tension alone was present

(b) If a net compressive bending stress is developed on a portion of the section, the member shall be proportioned so that F_{bc}, the allowable compressive bending stress (Clause 13.5) is not exceeded.

13.9 Load Bearing

The bearing stress, F_p, developed by a member or portion of a member, subjected to bearing shall be taken as:

(a) On the contact area of machined, accurately sawn or fitted parts

$F_p = 0.90F_y$

(b) On expansion rollers or rockers

$F_p = 0.00008F_y^2$

where

F_p is in newtons

D and L are the diameter and length respectively of roller or rocker

F_y is the specified minimum yield point of the weaker part in contact

(c) In bearing-type connections

$F_p = 1.5F_u$

where

F_u is the specified tensile strength of the plate

The ratio of end distance to bolt diameter shall meet the requirements of Clause 22.8.

13.10 Bolts in Bearing-Type Connections

13.10.1 General

For bolts in bearing-type connections Clause 13.10 ensures that the connection will not fail before the member.

13.10.2 Bolts in Shear

Stresses apply to bolt area determined from nominal bolt diameter.

(a) A307 bolts	70 MPa
(b) A325M and A325 bolts	205 MPa
(c) A490M and A490 bolts	275 MPa

When the bolt threads are intercepted by any shear plane, the allowable shear stress of any joint shall be taken as 70 per cent of F_v.

For joints longer than 1300 mm the shearing resistance shall be taken as 80 per cent of the above values.

13.10.3 Bolts in Tension

Stresses apply to bolt area determined from nominal bolt diameter.

(a) A307 bolts	140 MPa
(b) A325M and A325 bolts	300 MPa
(c) A490M and A490 bolts	370 MPa

13.10.4 Bolts in Combined Shear and Tension

Bolts subject to combined shear and tension due to forces applied to the connected parts shall be proportioned so that

$$\frac{f_t^2}{\alpha^2} + \frac{f_v^2}{\beta^2} \leqslant 1.0$$

where

α = permissible tensile stress as given in Clause 13.10.3

β = permissible shear stress as given in Clause 13.10.2

13.11 Bolts in Slip-Resistant (Friction-Type) Connections

13.11.1 General

The requirement for a slip-resistant connection is that under the forces and moments produced by specified loads, slip of the assembly shall not occur. In addition the allowable shear stresses determined in Clause 13.11 shall not exceed the applicable allowable stresses determined in Clause 13.10.2.

13.11.2 Shear Connections

The allowable stress, F_v, of a bolted joint, subjected to shear, shall be taken as:

$$F_v = 0.26\mu F_u$$

where μ, a function of slip probability, is expressed as a function of bolt type and condition of the faying surfaces of the parts. Representative values corresponding to a 5 percent probability of slip are given in Table 4.

Table 4
Values of μ

Category	**Steels**		**Steel Surface Treatment***	**Bolts**	
	CSA G40.21M	**ASTM**		**A325M A325**	**A490M A490**
1	Carbon and low-alloy steels except quenched and tempered		Tight clean mill scale		
2	300W, 300WT	A36	Vinyl wash primer	0.59	0.51
3	300W, 300WT	A36, A441	Blast-cleaned, organic zinc rich paint		
4			Blast-cleaned		
5			Blast-cleaned, inorganic zinc rich paint	0.99	0.87
6			Blast-cleaned, metallized		
7		A514	Blast-cleaned	0.69	0.60
8	300W, 300WT	A36, A441	Hot-dipped galvanized	0.31	0.27
9			Hot-dipped galvanized then wire-brushed or blasted	0.76	0.66

* See Clause 23.3.4 for steel surface treatment requirements.

13.11.3 Connections in Combined Shear and Tension

Bolts subject to combined shear and tension due to forces applied to the connected parts shall be proportioned so that

$$\frac{f_t}{\alpha} + \frac{f_v}{\beta'} \leqslant 1.0$$

where

α = permissible tensile stress as given in Clause 13.10.3

β' = permissible shear stress as given in Clause 13.11.2.

13.12 Welds

13.12.1 The resistance of welded joints is dependent upon the strength of the electrode used. Conditions wherein the electrode is "matched" to the base metal are given in Table 5(a).

13.12.2 The permissible stresses for the weld metal and base metal shall be as shown in Table 5(b).

13.12.3 The compressive resistance of joints utilizing partial joint penetration groove welds shall be based on the effective throat area of the welds plus the area of the base metal fitted in contact bearing.

Note: Compression joints that depend upon contact bearing, when assembled during fabrication, shall have at least 75% of the entire contact area in full bearing and the separation at the edges of the joint shall not exceed 0.60 mm unless otherwise stipulated by the Engineer.

13.12.4 The vector sum of longitudinal and transverse shear loads shall not exceed the stresses given in Table 5(b), unless a strength analysis is used that is acceptable to the Engineer.

13.12.5 Plug and slot welds shall be considered only to provide shear resistance in the plane of the connected parts.

Table 5(a)

Base Metal and Matching[1] Electrode Ultimate Strengths

Matching Electrode[2] Ultimate Strengths (MPa)	G40.21M Grades						
	260	300	350	380	400	480	700
410	X	X[4]					
480	X	X	X[3]	X			
550					X		
620						X	
820							X

Notes:

(1) For matching condition of ASTM steels see Table 11-1 or Table 12-1 of CSA Standard W59.

(2) Ultimate strengths have been determined from electrode classification numbers.

(3) For unpainted applications using "A" or "AT" steels where the deposited weld metal shall have similar atmospheric corrosion resistance and/or similar colour characteristics to the base metal, the requirements of Clause 5.2.1.4 and 5.2.1.5 of CSA Standard W59 shall apply.

(4) For HSS only.

Table 5(b)
Allowable Stresses for Welds

Type of Weld	Type of Load and Orientation	Requirements for matching*	Allowable Stress
Complete Joint Penetration Groove Welds	Tension or compression parallel to axis of weld*	Matching condition, or if electrode classification is lower or higher than matching. See note (2).	Same as for base metal
	Tension normal to effective throat	Matching is mandatory.	
	Compression normal to effective throat	Matching is mandatory.	
	Shear on effective throat	Matching condition	The smaller of (a) base metal $F_v = 0.40F_y$ or (b) weld metal $F_v = 0.30X_u$
		Electrode classification lower than matching. See note (1).	
Partial Joint Penetration Groove Welds	Tension or compression parallel to axis of weld*	Matching condition, or if electrode classification is lower or higher than matching. See note (2).	Same as for base metal
	Compression normal to axis of weld	Matching condition is mandatory.	See Note (4)

Table 5(b) continued

Type of Weld	Type of Load and Orientation	Requirements for matching*	Allowable Stress
Partial Joint Penetration Groove Welds	Tension normal to axis of weld	Matching condition, or if electrode classification is lower than matching use X_u of lower electrode.	The smaller of (a) base metal (lesser of) (i) $F_t = 0.60F_y$ (ii) $F_t = 0.50F_u$ or (b) weld metal $F_v = 0.30X_u$
	Shear	Matching condition, or if electrode classification is lower than matching use X_u of lower electrode	The smaller of (a) base metal $F_v = 0.40F_y$ b) Weld metal $F_v = 0.30X_u$
Fillet Welds	Tension or compression parallel to axis of weld	Matching condition, or if electrode classification is lower than matching. See Note (2).	Same as calculated for for base metal
	Tension or compression normal to axis of weld	For matching condition use X_u of matching electrode classification. For electrode classification lower than matching use X_u of lower electrode. For electrode classification one designation higher than matching use X_u of matching electrode.	The smaller of (a) base metal $F_v = 0.40F_y$ or (b) weld metal $F_v = 0.30X_u$
	Shear		
Plug and Slot Welds	Shear		

* For CSA G40.21M steels and their matching electrode classification refer to Table 5(a).

Notes:
1. Electrodes of strength lower than that shown for the corresponding base metal grades of Table 5(a) may be used for complete joint penetration groove welds between webs and flanges of girders transferring shear loads.

Table 5(b) continued

2. If shear is transferred between components, the design value is the smaller of that for the base metal or the weld metal.

3. For HSS, the provisions of Appendix L of CSA Standard W59, Welded Steel Construction (Metal Arc Welding) may be used.

4. Capacity of a joint is calculated as for base metal but on an area, $A = A_m$ + area of base metal in contact bearing.

5. The following areas are to be used in conjunction with the allowable stresses given in Table 5(b) to calculate permissible capacities,

A_w = the effective area of weld in shear
A_w shall be taken as follows:
- (a) For plug and slot welds: A_w = area of faying surface
- (b) For all others: A_w = size of effective throat of weld x length of weld

A_m = the area of fusion face
A_m shall be taken as follows:

- (a) For fillet welds: A_m = effective size x length of weld

- (b) For complete joint penetration groove welds:
  - (i) Butt Joint
    A_m = thickness of base metal x length of weld
  - (ii) T-Joint
    A_m = size of fusion face in base metal x length of weld

- (c) For partial joint penetration groove welds:
  - (i) Joints in Tension
    A_m = applicable area of base metal normal to tensile load
  - (ii) All Others, the lesser of:
    1. A_m = size of fusion face in base metal x length of weld
    2. A_m = thickness of base metal x length of weld

- (d) For Plug and Slot Welds: A_m = area of faying surface

F_u = specified ultimate tensile strength of base metal

F_y = specified minimum yield stress, yield point or yield strength of base metal

F_v = allowable stress

14. Cold Formed Steel Structural Members

14.1 Scope

14.1.1 Clause 14 shall apply to the design of structural members, cold formed to shape from carbon or low-alloy, sheet or strip steels, used for load-carrying purposes in buildings.

14.1.2 Where the provisions of Clause 14 differ from the provisions of other clauses of this Standard the provisons of Clause 14 shall govern for cold formed steel structural members.

14.2 Definitions and Symbols

14.2.1 **Definitions**
The following definitions apply to Clause 14.

Cold forming means the shaping of flat rolled steel at ambient temperature to form a structural section;

Effective width ratio means the ratio of the effective width (b) to the thickness (t) of the element. Effective width ratio shall be determined in accordance with requirements of Clause 14.4.4.2;

Flange of a section in bending means the flat width including any intermediate stiffeners plus the adjoining corners;

Flat width ratio means the ratio of the flat width (w) to the thickness (t) of the element;

Multiple stiffened element means an element that is adequately stiffened at both edges according to Clause 14.4.5.2 and also stiffened by means of intermediate stiffeners which are parallel to the direction of stress and which conform to the requirements of Clause 14.4.5.3;

Partially-effective element means an element for which the effective width is less than the flat width;

Point symmetric section means a section symmetric about its centroid;

Stiffened element means a flat element of which both edges parallel to the direction of stress are supported by stiffening means conforming to the requirements of Clause 14.4.4;

Sub-element of a multiple-stiffened element means the portion of such an element between adjacent stiffeners or between a web and intermediate stiffener or between an edge and intermediate stiffener;

Unstiffened element means a flat element with one longitudinal free edge;

Virgin steel means steel in the condition prior to cold forming (e.g., coiled or cut to length);

Web of a section in bending means that portion which joints two flanges. It is taken as the flat length measured in the plane of the web excluding the corners.

14.2.2 **Symbols**

The following symbols apply to Clause 14. Other symbols found in Clause 14 have previously been defined in Clause 5.

A_e	Effective cross-sectional area
a	Distance between web centerlines; distance between attachments
B	Force in bracing; stud spacing
B_L	Limiting value of B
b	Effective design width; distance between flange centerlines; width of largest leg of an angle
C_1	Factor used in calculating shear strain in sheathing of wall studs
c	Allowable amount of curling; distance from the centroid of a member to its extreme compressive fibre
D	Ratio of mean diameter to thickness of hollow circular section
D_A	Number of 90° corners in the flange of a section in bending or in the entire cross-section of a compression or tension member. If angles other than 90° are used, D_A is the sum of the bend angles divided by 90°
d	Clear distance between flanges
d_ℓ	Overall depth of lip

E	Modulus of elasticity for cold formed steel (203 000 MPa assumed)
E'	Modified Young's modulus used in wall stud load capacity equation
E_1	Term used in calculating shear strain in sheathing of wall studs
e	Normal distance from the hole center to the edge
F'	Compressive crushing stress
F_a	Allowable compressive stress under concentric loading
F_{be}	Buckling stress of a beam
F_{bw}	Critical web bending stress
F_c	Compressive stress in laterally unbraced single-web beams
F_e	Euler elastic buckling stress
F_p	Reduced elastic buckling stress
F_s	Euler elastic buckling stress about axis of symmetry
F_{st}	Torsional-flexural elastic buckling stress
F_t	Torsional elastic buckling stress
F_{tQ}	Stress used for wall stud load capacity calculation
F_u	Ultimate tensile strength of virgin steel
F_y	Tensile yield strength of virgin steel
F'_y	Average tensile yield strength which incorporates the effects of cold work of forming
F_{yf}	Tensile yield strength of flats
f	Calculated stress in an element
G	Shear modulus for cold formed sections (78 000 MPa assumed)
G'	Modified shear modulus used in wall stud load

capacity equation

g Distance of the fastener to the flanges that are tending to separate

I_s Moment of inertia of the full area of a multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis

I_{yc} Moment of inertia of cross-section of wall stud about the centroidal axis parallel to the web; moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web

M_y Moment causing a maximum strain of ε_y

m Distance of shear centre of channel from midplane of the web

N Actual length of bearing

P Force transmitted by bolt

P_L Lateral force used to design bracing

P_s Design load on wall studs

p Perimeter length of multiple stiffened element between edge stiffeners

q Uniformly distributed load on a beam, N/mm

$\bar{q}$ Shear rigidity per unit length spacing based on sheathing on both sides of stud

q_o Factor used to determine shear rigidity

R Term in effective width thickness ratio equation

r Inside bend radius; radius of gyration of the unreduced cross section for cold-formed sections

r_o Polar radius of gyration of cross-section about the shear center

S_{xc} Compressive section modulus of entire section about the major axis

s Spacing

T_s Allowable tensile load on connection

t Thickness of thinnest connected sheet

t_s Equivalent thickness

t_w Effective throat thickness of a fillet weld based on minimum leg size

W^* Ratio of the centerline length of the flange of a member in bending or of the entire section of a tension or compression member to the thickness

w Width of an element exclusive of fillets; flat width

w_f Projection of flange from inside face of web of a channel or half the distance between webs for box or U-type sections

w' Width of flange projection beyond the web for I-beam and similar sections or half the distance between webs for box or U-type sections; for flanges of I-beams and similar sections stiffened by lips at the outer edges, w' shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

x_o Distance from shear center to centroid of section

γ Shear strain in sheathing

$\bar{\gamma}$ Allowable shear strain in sheathing under load

ε Yield strain

μ Poisson's ratio (0.33 assumed)

θ Angle between web and plane of bearing surface

σ Allowable stress related to shear strain in wall studs

Φ Post buckling factor

ϕ_c Resistance factor for connections

ϕ_o Resistance factor for web crippling in beams having other than a single unreinforced web

ϕ_u Resistance factor for web crippling in beams with a single unreinforced web

14.3 Material Standards

14.3.1 General

Acceptable material and product standards and specifications for use under this clause are listed in Clauses 14.3.2 and 14.3.3.

14.3.2 Structural Steel

CSA G40.21-M,
Structural Quality Steels;

ASTM A36,
Structural Steel;

ASTM A242,
High Strength Low-Alloy Structural Steel;

ASTM A283,
Low and Intermediate Tensile Strength Carbon Steel Plates of Structural Quality;

ASTM A572,
High-Strength Low-Alloy Columbium-Vanadium Steels Structural Quality;

ASTM A588,
High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick;

14.3.3 Sheet Steel

ASTM A446M,
Steel Sheet, Zinc Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality (Grades A, B, C, D, F);

ASTM A570,
Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality;

ASTM A606,
Sheet Steel and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy with Improved Corrosion Resistance;

ASTM A607,
Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy Columbium and/or Vanadium;

ASTM A611,
Steel, Cold-Rolled Sheet, Carbon, Structural (Grades A, B, C & D);

ASTM A715,
Sheet Steel and Strip, Hot-Rolled, High-Strength Low-Alloy, with Improved Formability;

ASTM A816M,
Specification for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, High Strength Low-Alloy;

CSSBI 101-M,
Zinc Coated Structural Quality Steel Sheet for Roof and Floor Deck.

14.3.4 Physical Properties

The physical properties used for design purposes in Clause 14 shall be taken as:

Young's modulus E	203 000 MPa
Shear modulus G	78 000 MPa
Poisson's ratio (μ)	0.33
Mass density	7 850 kg/m^3
Coefficient of linear thermal expansion	11.7×10^{-6}/°C

14.4 General Design Considerations

14.4.1 Cold Work of Forming

14.4.1.1 General

Utilization of cold work of forming is optional and if used shall be in accordance with Clause 14.4.1.2, and only applied to the following additional Clauses of this Standard:

(a) Clause 14.5.3, Flexural Members - Single Web;

(b) Clause 14.5.5, Concentrically Loaded Compression Members;

(c) Clause 14.5.6, Members Subject to Combined Axial Load and Bending;

(d) Clause 14.5.8, Wall Studs.

14.4.1.2 Axially Loaded Tension Members; Tension Flanges of Beams; Fully Effective Axial Compression Members; Fully Effective Compression Flanges of Beams

The yield strength (F'_y) of (a) axially loaded tension members, (b) the tension flanges of flexural members, (c) fully effective axially loaded compression members and (d) the compression flanges of flexural members whose stiffened elements are not subject to a reduction in effective area as required by Clause 14.4.4.2, shall be determined by one of the following methods:

(a) Full section tensile tests as specified in Clause 14.8.3.1;

(b) $F'_y = F_y + 5D_A (F_u - F_y)/W^*$

14.4.1.3 **Axial Compression Members and Compression Flanges which are not Fully Effective**

The yield strength (F'_y) of axially loaded compression members and the compression flanges of flexural members not conforming to Clause 14.4.1.2 shall be taken as the tensile yield strength of the virgin steel specified by the relevant specification of Clause 5.

14.4.2 **Maximum Allowable Flat Width Ratios for Compression Elements**

Maximum allowable overall flat width ratios w/t, disregarding intermediate stiffeners and taking as t, the actual thickness of the element, unless otherwise specified shall be as follows:

(a) Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by:

(i) Simple lip bent at right angles to the element..............................60;

(ii) A more effective kind of stiffener.......90;

(b) Stiffened compression element with both longitudinal edges connected to a web or flange element (U-type or box-type sections).....................................500;

(c) Unstiffened compression element..............60.

Note: 1. Unstiffened compression elements (see Clause 14.4.2(c)) that have flat width ratios exceeding approximately 30 and stiffened compression elements (see Clause 14.4.2(b)) that have flat width ratios exceeding approximately 250 are likely to develop noticeable deformations under specified loads without detriment to the load-carrying ability.

2. Additional specific limits may be given in certain clauses.

14.4.3 Maximum Allowable Web Depths

The web slenderness ratio, h/t, of the webs of flexural members shall not exceed the following limitations:

(a) For unreinforced webs: $h/t \leq 200$;

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Clause 14.5.4

(i) When using bearing stiffeners only: $h/t \leq 260$;

(ii) When using bearing stiffeners and intermediate stiffeners: $h/t \leq 300$;

Where a web consists of two or more sheets, the ratio, h/t, shall be computed for the individual sheets.

14.4.4 Properties of Sections

14.4.4.1 General

Properties of sections such as cross sectional area, moment of inertia, section modulus and radius of gyration, shall be determined in accordance with conventional methods of structural design

14.4.4.2 Effective Width

When w/t exceeds $290\sqrt{\frac{k}{f}}$, w/t shall be replaced by an effective width ratio, b/t, as given in Clause 14.6.2.1. For stiffened compression elements, that portion of the total width which is considered removed to arrive at the effective width shall be located symmetrically about the centroid of the element. For unstiffened compression elements, the portion of the total width which is considered removed to arrive at the effective width shall be located at the unstiffened edge of the element.

14.4.4.2.1 Compression Elements and Sub-elements of Multiple-Stiffened Elements

The effective width ratio of compression elements, for strength determinations and for deflection or vibration determinations shall be taken as:

$$\frac{b}{t} = 428\sqrt{\frac{k}{f}}\left[1 - \frac{94}{(w/t)}\sqrt{\frac{k}{f}}\right] - R$$

Where

k = 4.0 for stiffened compression elements,
k = 0.5 for unstiffened compression elements,
f = stress in compression element computed on the

basis of the effective width.

R is given by:

(i) $R = 0.1\ (w/t) - 6$ when $(w/t > 60)$
(ii) $R = 0$ when $(w/t < 60$

When the element or sub-element is stiffened at each edge by means of a web or flange, R may be taken as zero for all values of w/t.

14.4.4.2.2 Effective Area of Stiffeners

For computing the effective structural properties of a member having compression sub-elements or elements subject to the correction factor R of Clause 14.4.4.2.1 in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area A_e as follows:

For $60 < w/t < 90$

$$A_e = \left(3 - 2(b/w) + (b/t)/30 - (w/t)/30\right)A$$

For $w/t > 90$

$$A_e = (b/w)A$$

Where A_e and A refer to the stiffener and w and b refer to the member.

14.4.4.2.3 Unusually Short Spans Supporting Concentrated Loads

Where the span of a flexural member is less than 30w', and the member carries one concentrated load, or several loads spaced farther apart than 2 w', the effective width of any flange, whether in tension or compression, shall be limited by the ratio given in Table 6.

Table 6
Short Wide Flanges
Maximum Allowable Ratio of Effective Width

L/w'	**30**	**25**	**20**	**18**	**16**	**14**	**12**	**10**	**8**	**6**
Ratio b/w	1.00	0.96	0.91	0.89	0.86	0.82	0.78	0.73	0.67	0.55

where L = full span for simple spans; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.

w' = width of flange projection beyond the web for I-beam and similar sections or half the distance between webs for box or U-type sections. For flanges of I-beams and similar sections stiffened by lips at the outer edges; w' shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

14.4.5 Stiffeners for Compression Elements

14.4.5.1 General

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener. See Clause 14.4.4.2.2 for effective area of stiffener.

14.4.5.2 Edge Stiffeners

In order that a flat compression element may be considered a stiffened compression element, it shall be stiffened along one longitudinal edge parallel to the direction of stress by a web, and along the other edge by a web, lip or other stiffener with a moment of inertia equal to the greater of

(i) $I = 9t^4$
(ii) $I = \left(2(w/t) - 13\right)t^4$

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required overall depth of the lip shall be taken as

$d_\ell = t\left(24(w/t) - 156\right)^{1/3}$ but not less than 4.8t.

A simple lip shall not be used as an edge stiffener for any element having a flat width ratio greater than 60.

14.4.5.3 Intermediate Stiffeners for Multiple-Stiffened Elements

In order that a flat compression element may be considered to be a multiple-stiffened element, it shall be stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners parallel to the direction of stress. Each such intermediate stiffener shall have a moment of inertia equal to twice that specified for edge stiffeners in Clause 14.4.5.2. Furthermore,

(a) If the spacing of stiffeners between two webs is such that the flat width ratio (w/t) of any of the sub-elements between stiffeners is larger than b/t only two intermediate stiffeners (those nearest each web) shall be considered effective;

(b) If the spacing of stiffeners between a web and an edge stiffener is such that the flat width ratio, w/t, of any of the sub-elements between stiffeners is larger than b/t, only the intermediate stiffener nearest the web shall be considered effective; and

(c) If intermedfiate stiffeners are spaced so closely that the flat width ratio, w/t between stiffeners does not exceed b/t, all the stiffeners may be considered effective. For this case, the flat width ratio and effective width ratio of the entire multiple-stiffened element, shall be determined using a width equal to the total width between webs or from web to edge stiffener, and a thickness equal to

$$t_e = t\left[\frac{w}{2p} + \sqrt{\frac{3I_s}{pt^3}}\right]^{1/3}$$

14.4.6 **Curling of Flanges**

Where a flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange towards the neutral axis, the gross width w/t of the flange shall be as follows:

(a) For tension flanges, unstiffened compression flanges and fully effective stiffened compression flanges:

$$w/t < \frac{450}{\sqrt{F_y}}\left(\frac{dc}{t^2}\right)^{1/4}$$

(b) For stiffened compression flanges that are not fully effective:

$$w/t < \frac{225}{t}\sqrt{\frac{dc}{F_y}}$$

where w is the gross width of flange projecting beyond the web or half the distance between webs for box or U-type beams.

Note: The allowable amount of curling will vary with different kinds of sections and must be established by the designer. (An amount of curling in the order of 5 per cent of the depth of the section is usually not considered excessive.)

14.5 Member Strength

14.5.1 General

To meet the strength requirements of this clause all allowable stresses shall be greater than or equal to all applied stresses on cold formed steel members as determined in clause 7.2. The factors of safety have been taken as follows:

(a) for tension, compression and shear flexure.......................... = 1.67

(b) for web crippling in beams:
  (i) Single unreinforced web............. = 2.0
  (ii) Other webs......................... = 2.5

(c) for connections......................... = 2.5

14.5.2 Axial Tension

14.5.2.1 The allowable tensile stress, F_t, developed by a member subjected to an axial tensile force shall be as given in Clause 13.2(a).

14.5.2.2 For angles connected by bolts in one leg the net section shall be reduced by 0.70 of the area of the outstanding leg. For channels connected by bolts in the web the net section shall be reduced by the area of the outstanding flanges.

14.5.3 Flexural

14.5.3.1 General

The average compressive stress, f_b, on extreme fibres of laterally unsupported straight flexural members shall not exceed

$$F_b = 0.60F_c$$

Where continuous lateral support is provided, $F_c = F_y$. Where continuous lateral support is not provided, F_c shall be computed in accordance with Clause 14.5.3.2 or 14.5.3.3.

14.5.3.2 Single-Web Members (I, Z, or Channel Shaped Members)

14.5.3.2.1 When bending is about the centroidal axis perpendicular to the web for either I-shaped sections or symmetrical channel-shaped sections, F_c is as follows:

(a) When $0.833(F_{be} + F_t) > 0.5F'$

$$F_c = F' - \left[\frac{0.25(F')^2}{0.833(F_{be} + F_t)}\right] \text{ but not greater than } F_y$$

(b) When $0.833\ (F_{be} + F_t) < 0.5F'$

$$F_c = 0.833(F_{be} + F_t)$$

where

$F_{be} = \pi^2 EdI_{yc}/\omega L^2 S_{xc}$

$F_t = 0.333GAt^2/\omega dS_{xc}$

$F' = 1.11F_y$

ω see Clause 13.7.4

14.5.3.2.2 For point-symmetrical Z-shaped sections bent about the centroidal axis perpendicular to the web:

(a) When $0.833\ (F_{be} + F_t) > 0.5F'$

$$F_c = F' - \left[\frac{0.5(F')^2}{0.833(F_{be} + F_t)}\right] \text{ but not greater than } F_y$$

(b) When $0.833(F_{be} + F_t) < 0.5F'$

$$F_c = 0.833(0.5F_{be} + 0.5F_t)$$

14.5.3.3 **Closed Box Flexural Members**

When bending is about the major axis of the section, F_c shall be taken as:

$$F_c = \frac{\pi}{LS_{xc}} \sqrt{EI_y GJ}$$

where

J = St. Venant torsion constant, $= \dfrac{2(ab)^2}{(a/t_1) + (b/t_2)}$

where

a = distance between web centrelines
b = distance between flange centrelines
t_1 = thickness of flanges
t_2 = thickness of webs

14.5.3.4 **Bending Stresses in Webs**

The maximum bending stress shall not exceed F_y in tension or compression and in addition the maximum bending stress in compression shall not exceed:

$$F_c = \Phi F_{bw}$$

where

$$F_{bw} = \frac{183\ 000k}{(h/t)^2}$$

$$\Phi = \alpha_1\alpha_2\alpha_3\alpha_4 \geq 1.0$$

$$\alpha_1 = 0.017(h/t) - 0.79$$

$$\alpha_2 = 0.462 \left|\frac{f_c}{f_t}\right| + 0.538$$

For beams with stiffened compression flanges

$$\alpha_3 = 1.16 - 0.16 \frac{(w/t)}{290\sqrt{k/F_y}} \leq 1 \text{ when } \frac{(w/t)}{290\sqrt{k/F_y}} \leq 2.25$$

$$\alpha_3 = 0.8 \text{ when } \frac{(w/t)}{290\sqrt{k/F_y}} > 2.25$$

For beams with unstiffened compression flanges

$$\alpha_3 = 0.84 - 0.019 \frac{(w/t)}{290\sqrt{k/F_y}}$$

$$\alpha_4 = \left(\frac{F_y}{406}\right) + 0.10$$

$$k = 4 + 2\left(1 + \left|\frac{f_t}{f_c}\right|\right)^3 + 2\left(1 + \left|\frac{f_t}{f_c}\right|\right)$$

f_t = maximum tensile bending stress in web

f_c = maximum compressive bending stress in web

14.5.3.5 **Shear in Webs**

The allowable shear stress F_v developed by the web of a flexural member subjected to shear shall be taken as follows:

the lesser of:

(a) $F_v = 0.40F_y$

(b) (i) For $h/t \leq 618\sqrt{\frac{k_v}{F_y}}$ .... $F_v = \frac{178\sqrt{k_v F_y}}{(h/t)}$

(ii) For $h/t > 618\sqrt{\frac{k_v}{F_y}}$ ...... $F_v = \frac{107\ 000\ k_v}{(h/t)^2}$

where

A_w = ht as defined in Clause 13.4.1

Where the web consists of two or more sheets, each sheet shall be considered to carry its share of the shear.

14.5.3.6 **Combined Bending and Shear Stresses in Webs**

For webs subject to both bending and shear stresses, the member shall be proportioned such that the following limits are observed:

$$\frac{f_b}{F_b} < 1$$

$$\frac{f_v}{F_v} < 1$$

$$\left(\frac{f_b}{F_b}\right)^2 + \left(\frac{f_v}{F_v}\right)^2 < 1.0$$

For beam webs with transverse stiffeners satisfying the requirements of Clause 14.5.4 the member may be proportioned such that the following limits are observed:

$$\frac{f_b}{F_b} < 1$$

$$\frac{f_v}{F_v} < 1$$

$$0.6\left(\frac{f_b}{F_b}\right) + \left(\frac{f_v}{F_v}\right) < 1.3 \quad \text{when} \quad \frac{f_b}{F_b} > 0.5$$

$$\text{and} \quad \frac{f_v}{F_v} > 0.7$$

14.5.3.7 **Web Crippling**

Bearing stiffeners shall be provided when h/t > 200 and when the applied concentrated load or reaction exceeds the allowable bearing load for the webs as given in Tables 7, 8, and 9 where P is the resistance offered by one solid web. The bearing capacity of two or more sheets is the sum of the individual capacities.

For built-up I-members, or similar sections, the distance between the connector and member flange shall be kept as small as practicable.

Reactions or loads are classified as single flange loading when the clear distance measured longitudinally between the bearing edges of adjacent loads on opposite flanges exceeds 1.5h and as opposite flange loading when this distance is equal to or less than 1.5h.

Reactions or loads are classified as end reactions when the distance from the edge of the bearing to the end of the member is equal to or less than 1.5h and as interior reactions when this distance exceeds 1.5h.

The bearing resistance of two channels connected back to back and for similar sections which provide a high degree of restraint against rotation of the web, such as I-sections made by welding two angles to a channel shall be taken as given in Table 7.

In Tables 7, 8 and 9

$C_1 = (1.49 - 0.53k) \geq 0.6$

$C_2 = 1 + \frac{(h/t)}{750} \leq 1.2$

$C_3 = \frac{1}{k}$ when $h/t \leq 66.5$;

$C_3 = \left(1.1 - \frac{(h/t)}{665}\right)/k$ when $h/t > 66.5$

$C_4 = 0.98 - \frac{(h/t)}{865}/k$

$k = F_y/230$

θ = angle between plane of web and plane of bearing surface > 45° but no more than 90°

Table 7
I-Beams

Flange Loading	Reaction or Load	Allowable Load, P_a
Single	End	$P_a = C_2\left(5 + 0.63\sqrt{N/t}\right)t^2F_y$
	Interior	$P_a = C_1\left(0.88 + 0.063t\right)\left(7.5 + 1.63\sqrt{N/t}\right)t^2F_y$
Opposite	End	$P_a = C_4\left(0.64 + 0.16t\right)\left(5 + 0.63\sqrt{N/t}\right)t^2F_y$
	Interior	$P_a = C_3\left(0.82 + 0.079t\right)\left(7.5 + 1.63\sqrt{N/t}\right)t^2F_y$

The above formulas apply when $\frac{r}{t} \leq 4$, $\frac{N}{t} \leq 200$ and $N/h \leq 1$

Table 8
Shapes Having Single Webs

Flange Loading	Reaction or Load	Allowable Load, P_a
Single	End	Stiffened Flanges $P_a = 5(1.33 - 0.33k)(1.15 - 0.15(r/t))(1 + 0.01(N/t))(1 - 0.0018(h/t))t^2F_y$
		Unstiffened Flanges $P_a = 3.3(1.33 - 0.33k)(1.15 - 0.15(r/t))(1 + 0.01(N/t))^{*}(1 - 0.0013(h/t))t^2F_y$
	Interior	$P_a = 8(1.22 - 0.22k)(1.06 - 0.06(r/t))(1 + 0.007(N/t))^{**}(1 - 0.0014(h/t))t^2F_y$
Opposite	End	$P_a = 3.7(1.33 - 0.33k)(1.15 - 0.15(r/t))(1 + 0.01(N/t))(1 - 0.0023(h/t))t^2F_y$
	Interior	$P_a = 8(1.22 - 0.22k)(1.06 - 0.06(r/t))(1 + 0.01(N/t))(1 - 0.0029(h/t))t^2F_y$

* When $N/t > 60$, the factor $(1 + 0.01N/t)$ may be increased to $(0.71 + 0.015N/t)$.

** When $N/t > 60$, the factor $(1 + 0.007N/t)$ may be increased to $(0.75 + 0.011N/t)$.

The above formulas apply when $r/t < 4$, $N/t < 200$ and $N/h < 1$.

Table 9
Deck Sections

Flange Loading	**Reaction or Load**	**Allowable Load, P_a**
Single	End	$P_a = 7(\sin\theta)(1 - 0.1k)(1 - 0.1\sqrt{r/t})$ $(1 + 0.005(N/t))(1 - 0.002h/t)t^2F_y$
	Interior	$P_a = 8.5(\sin\theta)(1 - 0.1k)(1 - 0.075\sqrt{r/t})$ $(1 + 0.005(N/t))(1 - 0.001h/t)t^2F_y$
Opposite	End	$P_a = 5.5(\sin\theta)(1 - 0.1k)(1 - 0.1\sqrt{r/t})$ $(1 + 0.01(N/t))(1 - 0.002(h/t))t^2F_y$
	Interior	$P_a = 9(\sin\theta)(1 - 0.2k)$ $(1 - 0.03\sqrt{r/t})(1 + 0.01(N/t))$ $= (1 - 0.0015(h/t))t^2F_y$

The above formulas apply to decks when $r/t < 10$ and $N/t < 200$.

Note: For hat sections both legs must be fastened to prevent spreading.

14.5.3.8 **Combined Web Crippling and Bending**
Unreinforced flat webs of shapes subjected to a combination of bending and web crippling shall be designed to meet the following requirements:

$$\frac{P}{P_a} + \frac{M}{M_a} < 1.3$$

This interaction equation is not applicable to multi-web deck sections.

14.5.4 **Transverse Stiffeners for Beam Webs**

14.5.4.1 **Bearing Stiffeners**
Bearing stiffeners shall bear against the flanges or flange through which they receive their loads. Stiffeners shall be designed as columns in accordance with Clause 16.5.1 assuming the column

section to comprise the pair of stiffeners and a centrically located strip of the web less than or equal to eighteen times its thickness at interior stiffeners, or strip not equal to more than 10 times its thickness when the stiffeners are located at the end of the web. The effective column length KL shall be taken as not less than 3/4 the length of the stiffeners. Bearing stiffeners shall be connected to the web in accordance with clause 14.6 so as to develop the full force required to be carried by the stiffener into the web or vice versa.

The flat width ratio, w/t, of the stiffened and unstiffened elements of transverse stiffeners shall not exceed $780/\sqrt{F_y}$ and $228/\sqrt{F_y}$, respectively.

14.5.4.2 Intermediate Stiffeners

Intermediate transverse stiffeners when used shall be spaced to suit the shear resistance determined from the formula given in Clause 14.5.3.5 and the maximum distance between stiffeners shall be as given in Clause 16.6.2.

Intermediate transverse stiffeners may be furnished singly or in pairs. The moment of inertia of the stiffener or pair of stiffeners if so furnished must be at least equal to the greater of

(i) $I = (h/50)^4$

(ii) $I = 5ht^3[h/a - 0.7(a/h)]$

taken about an axis in the plane of the web. The gross area of intermediate stiffeners, or pairs of stiffeners if so furnished, shall be taken as

$$A_s \geq \frac{at}{2}\left[1 - \frac{a/h}{(a/h) + \left(1 + (a/h)^2\right)^{1/2}}\right] CYD$$

where

a   distance centre to centre of adjacent stiffeners (i.e. panel length)

$C = 1 - \dfrac{310\ 000\ k_v}{F_y(h/t)^2}$ but not less than 0.10

Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel

D = stiffener factor
= 1.0 for stiffeners furnished in pairs
= 1.8 for siingle angle stiffeners
= 2.4 for single plate stiffeners

k_v = shear buckling coefficient (see Clause 13.4.1)

F_y = specified minimum yield point of web steel

The allowable load on stamped or rolled-in transverse stiffeners shall be determined by tests in accordance with Clause 14.8.

14.5.5 Axially Loaded Compression Members

14.5.5.1 General

The requirements of Clause 14.5.5 apply only to material of 4.5 mm or less in thickness. Members formed from thicker material shall be designed in accordance with the requirements of Clause 13.3. Allowable compressive stresses of singly-symmetric shapes are given in Clause 14.5.5.3; hollow structural sections are given in Clause 14.5.5.4; built-up members are given in Clause 14.5.5.6 and axially loaded wall studs are given in Clause 14.5.8. Except as given in Clause 14.5.5.5 in which $w/t \leq 150$ for stiffened compression elements and $w/t \leq 35$ for unstiffened compression elements F_a shall be taken as:

$$F_a = 0.60F_o$$

(a) For $F_p > F_y/2$ $\quad F_o = F_y - \dfrac{F_y^{\,2}}{4F_p}$

(b) For $F_p < F_y/2$ $\quad F_o = F_p$

where F_p is defined in Clauses 14.5.5.2 to 14.5.5.3.

14.5.5.2 Sections not Subject to Torsional-Flexural Buckling

For I-shapes, closed cross section shapes, and any other shapes which can be shown not to be subject to torsional-flexural buckling, F_p shall be taken as:

$$F_p = 0.833\pi^2E/(KL/r)^2$$

14.5.5.3 Singly-Symmetric Shapes

For singly-symmetric open sections, such as plain and lipped channels and single or double plain and lipped angles which may be subject to torsional-flexural buckling, F_p shall be taken as the lesser of:

(a) F_p as defined in Clause 14.5.5.2

(b) $F_p = \frac{1}{2\beta}\left\{F_s + F_t - \sqrt{(F_s + F_t)^2 - 4\beta F_s F_t}\right\}$

where:

$$F_s = 0.833\pi^2E/(KL/r_s)^2$$

$$F_t = \frac{0.833}{Ar_0^2}\left[GJ + \frac{\pi^2 EC_w}{(KL)^2}\right]$$

$$\beta = 1 - (x_0/r_0)^2$$

$$r_0^2 = r_x^2 + r_y^2 + x_0^2$$

KL = effective length with respect to torsion

x_0 = distance from shear centre to centroid along x-axis

$$J = \frac{1}{3}\sum \ell\ t^3$$

ℓ = middle line length of member segments

KL/r = maximum slenderness ratio with respect to the x or y axis

r_s = radius of gyration about the axis of symmetry

14.5.5.3.2 For channel, Z shape and single angle sections with unstiffened flanges shall be designed in accordance with Clauses 14.5.5.1 and 14.5.5.3 except that the allowable stress shall be limited additionally as follows:

$$F_a = \frac{45\ 700}{(w/t)^2}$$

This additional limit shall be waived if the channel is fully restrained with respect to torsion and flexural buckling about the asymmetric axis.

14.5.5.4 **Hollow Structural Sections**

The design of hollow structural section compression members that comply with the requirements of CSA Standard G40.20, General Requirements for Rolled or Welded Structural Quality Steel, shall meet the requirements of Clause 13.3.2.

14.5.5.5 **Other Sections**

For non-symmetric shapes whose cross-sections do not have any symmetry, either about an axis or a point, and for sections formed with any stiffened element whose flat width ratio exceeds 150 or any unstiffened elements whose flat width ratio exceeds 35, the compressive strength shall be determined by rational analysis. Alternatively, compression members composed of such shapes may be tested in accordance with Clause 14.8.

14.5.5.6 **Built-up Members**

14.5.5.6.1 For compression members composed of two or more elements connected together at discrete points, such as double angles and battened channels, subjected to buckling about the composite axis, F_p shall be taken as:

$$F_p = 0.833 \left[\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2 + \left(\frac{a}{r_z}\right)^2}\right]$$

where

KL/r = the overall slenderness ratio of the complete section about the composite axis

a/r_z = the slenderness of the individual elements between points of connection, about an axis parallel to the composite axis

14.5.5.6.2 Each discrete connection shall be capable of transmitting a longitudinal shear force between the elements, of 5 per cent of the force in one element.

14.5.5.6.3 For torsional-flexural buckling of singly-symmetric sections F_p shall be taken as given in Clause 14.5.5.3.1 with F_s replaced by F_p as given in Clause 14.5.5.6.1.

14.5.6 Combined Axial Load and Bending

14.5.6.1 Axial Compression and Bending

Members required to resist bending moments and an axial compressive force shall be proportioned so that:

(a) $$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

where $F_a = 0.60F_y$ and,

(b) $$\frac{f_a}{F_a} + \frac{\omega_x F_{bx}}{F_{bx}\left(1 - \frac{f_a}{F'_{ex}}\right)} + \frac{\omega_y F_{by}}{F_{by}\left(1 - \frac{f_a}{F'_{ey}}\right)} \leq 1.0$$

where

F_a is defined in Clause 15.5.5

F_b is defined in Clause 15.5.3

$$F'_e = \frac{1\,000\,000}{\left(\frac{KL}{r}\right)^2}$$

ω_x, ω_y = as defined in Clause 13.7.2

When $f_a/F_a \leq 0.15$ the value of $\left(1 - \frac{f_a}{F'_{ex}}\right)$, $\left(1 - \frac{f_a}{F'_{ey}}\right)$, ω_x and ω_y may be assumed to be equal to one.

14.5.6.2 Axial Tension and Bending

Members required to resist both bending moments and an axial tensile force shall be proportioned in accordance with Clause 13.8. The bending stress, F_b, shall be based on the net section.

14.5.7 Single Angles Loaded Through One Leg

For single angles loaded at each end through the same leg by bolts or welds, the compressive strength shall be as given in Clause 14.5.5.1 where

$$F_p = 0.833 \left[\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2 + \left(\frac{5b}{t}\right)^2} \right]$$

where r = minimum radius of gyration
b = leg width
K = 0.8 for single bolt connections and
0.7 for two bolt connections or welds

14.5.8 Wall Studs

The allowable compressive load on a stud may be computed on the basis that wall materials or sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided the stud, wall material, and attachments comply with the following requirements.

(a) Both ends of the stud shall be braced against rotations about the longitudinal stud axis and translations perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis.

(b) The cladding shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

14.5.8.1 Studs in Compression

14.5.8.1.1 For studs having identical sheathing material with design shear rigidity, $\bar{q}B$ attached to both flanges, and neglecting any rotational restraint provided by the sheathing material the compression resistance shall be taken as:

$$F_a = 0.60 F_o$$

where F_o is the least of:

(a) F_o as given in Clause 14.5.6.1 with KL equal to two times the distance between fasteners

(b) $F_o = F_y - \frac{F_y^2}{4F_p}$ when $F_p > F_y/2$

$F_o = F_p$ when $F_p \leq F_y/2$

(c) $F_o = 0.833\sigma$, where σ is determined to satisfy the requirement that the shear strain, γ, in the sheathing corresponding to the stress, σ, shall not exceed the allowable shear strain of the sheathing, $\bar{\gamma}$ given in Table 10.

$$\bar{\gamma} \geq \frac{\pi}{L}\left[C_1 + E_1 \frac{d}{2}\right]$$

where F_p is given in Clause 14.5.8.1.2 and C_1

and E_1 are given in Clause 14.5.8.1.3. To initiate the iterative calculations required to establish the strain compatibility of γ and $\bar{\gamma}$, σ should initially be taken as F_o as computed in Clause 14.5.8.1.1(b).

14.5.8.1.2 (a) For singly symmetric channel and C sections F_p shall be taken as the lesser of:

(i) $F_p = 0.833[F_{ey} + (\frac{\bar{q}B}{A})]$

(ii) $F_p = \frac{0.833}{2\beta}[(F_{ex}+F_{tQ}) - \sqrt{(F_{ex}+F_{tQ})^2 - 4\beta F_{ex}F_{tQ}}\ ]$

(b) For Z-sections F_p shall be taken as the lesser of:

(i) $F_p = 0.833[F_t + (\frac{\bar{q}Bd^2}{4Ar_o^2})]$

(ii) $F_p = \frac{0.833}{2}[(F_{ex} + F_{ey} + (\frac{\bar{q}B}{A})) - \sqrt{(F_{ex} + F_{ey} - (\frac{\bar{q}B}{A})) - 4(F_{ex}F_{ey} + F_{ex}(\frac{\bar{q}B}{A}) - F_{exy}^2)}\ ]$

(c) For doubly stymmetric I-sections, F_p shall be taken as the lesser of:

(i) $F_p = 0.833(F_{ey} + \frac{\bar{q}B}{A})$

(ii) $F_p = 0.833F_{ex}$

In the above formulas

$$F_{ey} = \frac{\pi^2 E}{(L/r_y)^2}$$

$$F_{ex} = \frac{\pi^2 E}{(L/r_x)^2}$$

$$F_{exy} = \frac{\pi^2 EI_{xy}}{AL^2}$$

$$F_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{L^2}\right]$$

$$F_{tQ} = F_t + \left(\frac{\bar{q}Bd^2}{4Ar_o^2}\right)$$

where

$\bar{q}$ = shear rigidity per mm of stud spacing based on sheathing on both sides for two wallboards per mm of stud given in Table 10
B = stud spacing
A = cross sectional area of stud
β = $1 - (x_o/r_o)^2$
x_o = distance from shear center to centroid along principal x-axis
r_o = polar radius of gyration of cross-section about shear center

14.5.8.1.3 The values for C_1 and E_1 shall be taken as follows

(a) For Single Symmetric Channel and C Sections

$$C_1 = \frac{\sigma C_o}{F_{ey} - \sigma + \bar{Q}_a}$$

$$E_1 = \frac{\sigma[(F_{ex} - \sigma)(r_o^2 E_o - x_o D_o) - \sigma x_o (D_o - x_o E_o)]}{(F_{ex} - \sigma)\ r^2\ (F_{tQ} - \sigma) - (\sigma x_o)^2}$$

(b) For Z Sections

$$C_1 = \frac{\sigma[C_o(F_{ex} - \sigma) - D_o F_{exy}]}{(F_{ey} - \sigma + \bar{Q}_a)(F_{ex} - \sigma) - (F_{exy})^2}$$

$$E_1 = \frac{\sigma E_o}{F_{tQ} - \sigma}$$

(c) For I Sections

$$C_1 = \frac{\sigma C_o}{F_{ey} - \sigma + \bar{Q}_a}$$

$$E_1 = 0$$

where

Table 10
Sheathing Parameters[1]

Wall Board[2]	$\bar{q}_o$ [3] N/mm	$\bar{\gamma}$ mm/mm
9.5 to 15.9 mm thick gypsum	525	0.008
Lignocellulosic board	263	0.009
Fiberboard (regular or impregnated)	158	0.007
Fiberboard (heavy impregnated)	315	0.0010

Notes:

(1) The values given were established from small-scale tests and are subject to the following important limitations: All values are for wall boards on both sides of the wall assembly. All fasteners are No. 6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent, at 150 to 300 mm spacing.
(2) All wall boards are 12.7 mm thick, except as noted.
(3) $\bar{q} = \bar{q}_o(2 - s/300)$
where s = the fastener spacing

For other types of claddings, $\bar{q}_o$ and $\bar{\gamma}$ may be determined conservatively from representative small-specimen tests as described by published documented methods. Wall board parameter values $\bar{q}_o$ and $\bar{\gamma}$, determined from representative full-scale tests described by published documented methods, may also be used instead of the small-scale test values given in Table 10.

C_o, E_o, and D_o are initial column imperfections which shall be assumed to be at least

C_o = L/350 in a direction parallel to wall

D_o = L/700 in a direction parallel to wall

E_o = L/(d x 10 000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled location

In case $\sigma > 0.5F_y$, then in the definitions for F_{ey},

F_{ex}, F_{exy}, and F_{tQ}, the parameters E and G are to be replaced in Clause 14.5.8.1.1(ii) by E' and G', respectively, given as:

$$E' = 4E_o(F_y - \sigma)/(F_y)^2$$

$$G' = G(E'/E)$$

14.5.8.1.3 Studs with sheathing on one side only; or unidentical sheathing; or when the rotational restraint is not neglected; or any combination of the above shall be designed in accordance with the same basic principles of analysis used in deriving the provisions in Clause 14.5.8.1.1.

14.5.8.2 **Studs in Axial Compression and Bending**

The design strength of studs subject to combined axial compression and bending shall be determined by:

$$\frac{f_a}{F_a} + \frac{f_b}{\left[1 - \frac{f_a}{F'_a}\right] F_b} < 1.0$$

when $\frac{f_a}{F_a} < 0.15$, the following formula may be used in lieu of the above:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$$

where

F_a = compression resistance under concentric loading according to Clause 14.5.5

M_b = bending resistance where bending only exists (Clauses 14.5.2 and 14.5.3),

$F'_a = 0.6[\pi^2 E/(L/r_x)^2]$

14.6 Connections

14.6.1 Fastening Devices

Any suitable mechanical fastener, special device, or other means may be used to join component parts provided that the type of fastening device is compatible with the service conditions.

14.6.2 Welded Connections

14.6.2.1 General

An allowance shall be made for the effect of welding on the mechanical properties of the member. This effect shall be determined by tests on full section specimens which contain the weldment within the gauge length.

14.6.2.2 Arc Welds

14.6.2.2.1 General Requirements

Arc welds shall meet the following requirements depending on the minimum thickness, t, of the connected parts.

(a) $t > 3.5$ mm
Fusion welds on steel shall conform to the requirements of CSA Standard W59, "Welded Steel Construction (Metal-Arc Welding)".

(b) $0.70 \text{ mm} \leq t \leq 3.5$ mm
Fusion welds on steel shall conform to the requirements contained herein, and in all cases the welding shall be performed in accordance with CSA Standard W59.

(c) $t < 0.70$ mm
Fusion welds made on steel shall be considered to have no structural value.

14.6.2.2.2 Butt Welds

The resistance of butt welds in tension or compression shall be taken as the lower strength of base metal joined. The weld shall have full penetration.

14.6.2.2.3 Arc-Spot Welds (Puddle Welds)

Arc spot welds shall be used only for the welding in the flat position of sheet steel to a supporting member. Type E410XX or E480XX electrodes shall be used to melt through the sheet steel to fuse with the plate. The weld shall be round in shape with a visible nominal diameter of 20 mm. The thickness of the structural supporting member shall be at least 2.5 times the steel sheet thickness. The minimum edge distance measuring from the centerline of the weld to the end or boundary of the connected member shall not be less than 25 mm. The steel sheet shall be of weldable type and shall have a tensile yield strength of 230 MPa or greater. The allowable shear load per weld shall be taken as:

(a) For shear

$$V = 400(20t - 5)$$

(b) For tension

$T = 400(5.6t - 1)$

Note: The resistances apply only to sheet thickness from 0.70 mm to 1.52 mm.

14.6.2.2.4 Fillet Welds

Fillet welds may be made in any position. The allowable shear load for fillet welds shall be taken as:

(a) For welds parallel to the direction of loading

When $L/t < 25$ $\quad V = 0.4\left(1 - 0.01\,\frac{L}{t}\right) tLF_u$

When $L/t \geq 25$ $\quad V = 0.3tLF_u$

(b) For welds perpendicular to the direction of loading

$V = 0.40tLF_u$

14.6.2.2.5 Flare Groove Welds

Flare groove welds may be made in any welding position. Sheet to sheet connections may be made with flare-V and flare-bevel groove welds and sheet to thicker steel member connections may be made with flare-bevel groove welds. The factored shear resistance of welds shall be governed by the thickness, t, of the sheet steel adjacent to the welds. The allowable shear load shall be taken as:

(a) For loads applied perpendicular to the axis of the weld

(i) Flare-bevel groove welds

$V = 0.32tLF_u$

(ii) Flare-V groove welds

Loads applied perpendicular to the axis of the weld have not been considered.

(b) For loads applied parallel to the axis of the weld

when $t \leq t_w < 2t$ or $d_\ell < L$

$V = 0.30tLF_u$

when $t_w \geq 2t$ or $d_\ell > L$

$V = 0.60tLF_u$

where t_w is the lesser of the two throats shown in Figure 1.

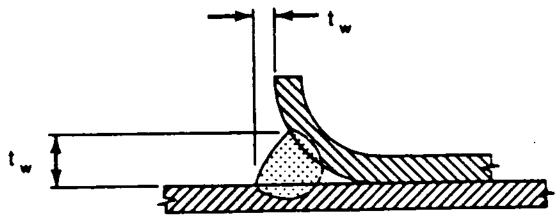


Figure 1.

14.6.2.3 **Resistance Welds**
The allowable shear strength per spot weld for sheets joined by spot welding shall be taken as

$$V_r = 1600t^{1.5}$$

This equation applies to welds in sheets between 0.40 mm and 6 mm thick.

14.6.3 **Connections Made by Bolts, Rivets and Screws**

14.6.3.1 **Shear Strength**

14.6.3.1.1 The shear strength for bolts shall be taken as given in Clauses 13.10 and 13.11.

14.6.3.1.2 For screws and special fasteners for which Clause 14.6.3.1.1 cannot be applied, the allowable strength shall be taken as the manufacturer's certified ultimate shear resistance in the condition specified divided by 2.5.

14.6.3.2 **Bearing Stress in Bolted Connections**

14.6.3.2.1 For bolted connections with washers under both bolt head and nut the bearing stress on the area, dt, shall not exceed the allowable stress as follows:

(a) For an inside sheet of double shear connection

(i) For $F_u/F_y > 1.15$ $\quad F_p = 1.50F_u$

(ii) For $F_u/F_y < 1.15$ $\quad F_p = 1.35F_u$

(b) For a single shear, and outside sheets of double shear connection

$$F_p = 1.35F_y$$

14.6.3.2.2 For bolted connections without washers under both bolt head and nut, or with only one washer where $0.91 \le t < 4.76$ the bearing stress on the area, dt, shall not exceed the allowable stress as follows:

(a) For an inside sheet of double shear connection

For $F_u/F_y \ge 1.15$ $F_p = 1.35F_u$

(b) For single shear, and outside sheets of double shear connection

For $F_u/F_y \ge 1.15$ $F_p = 1.00F_u$

14.6.3.2.3 When $t > 4.76$ mm Clause 13.10 shall be used. For conditions not defined in Clauses 14.6.3.2.1 and 14.6.3.2.2 stresses shall be determined on the basis of test data using a factor of safety of 2.5.

14.6.3.3 **Tension Stress on Net Section**

The net cross section may be considered to be fully effective in tension for symmetrically applied forces.

14.6.3.3.1 Clause 14.6.3.3 shall only apply when $t \le 4.76$ mm. When $t > 4.76$ mm Clause 13.10 shall apply.

14.6.3.3.2 The tension stress, F_t, on the net cross section of a bolted connection with washers under both head and nut shall not exceed $0.60F_y$ nor shall it exceed the following:

(a) For double shear connection F_t shall be taken as the lesser of

(i) $F_t = (1.0 - 0.9r + 3rd/s)0.50F_u$

(ii) $F_t = 0.50F_u$

(b) For single shear connection F_t shall be taken as the lesser of

(i) $F_t = (1.0 - 0.9r + 3rd/s)0.45F_u$

(ii) $F_t = 0.45F_u$

14.6.3.3.3 The tension stress, F_t, on the net cross section of a bolted connection without washers under bolt head and nut or with only one washer shall not exceed $0.60F_y$ nor shall it exceed the following:

(i) $F_t = (1.0 - r + 2.5rd/s)0.45F_u$

(ii) $F_t = 0.45F_u$

where r = the force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0.2, it may be taken equal to zero.

14.6.3.4 **Minimum Edge Distance and Spacing**
The distance from the centre of a fastener to the edge shall not be less than 1.5d. The distance between fasteners, centre-to-centre, shall not be less than 2.5d.

14.6.4 **Connections in Built-up Members**

14.6.4.1 The number of fasteners joining elements together to form a beam or column shall be sufficient to transfer the shear forces developed. The spacing of fasteners shall be such that buckling of the elements between fasteners is prevented.

14.6.4.2 The allowable compressive stress of built-up members shall be determined in accordance with Clause 14.5.5.6.

14.6.4.3 Fasteners in beams, at the point of application of concentrated loads shall be capable of transferring any such load applied to one element only.

The connection at points of application of concentrated loads of double channel beams shall be capable of resisting a force, T, tending to separate the flanges and shall be taken as:

$$T = \frac{Pm}{2g}$$

For double channel beams subject to a uniformly distributed load, P shall be taken as:

$$P = 3sq$$

Maximum fastener spacing for beams shall not exceed L/4.

14.6.5 **Spacing of Connections in Compression Elements**
The spacing, s, in line of stress, of welds, rivets, or bolts connecting a compression cover plate or sheet to a non-integral stiffener or other element shall not exceed

(a) the spacing required to transmit the shear between the connected parts based on connection strength

(b) $s = 680t/\sqrt{f}$

where f is the working stress in the cover plate or sheet

(c) $s = 3w$, or $500t/\sqrt{F_y}$, whichever is greater

where

w = flat width of the narrowest unstiffened compression element in the portion of the cover plate or sheet which is tributary to the connections.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds plus 13 mm. In all other cases the spacing shall be taken as the centre-to-centre distance between connections.

Note: The requirements of Clause 14.6.5 do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

14.7 Bracing

14.7.1 General

14.7.1.1 General requirements for the bracing of compression members and compression flanges of beams and compression chords of trusses are given in Clause 20.

14.7.1.2 The provisions of Clause 14.7.2 apply to compression members and flexural members of symmetric section in which the applied loading does not induce twist.

14.7.1.3 The provisions of Clause 14.7.3 apply to flexural members, such as channel and Z sections, in which load applied in the plane of the web induces twist.

14.7.2 Sections Which are Symmetric Relative to the Plane of Loading

14.7.2.1 Discrete Braces

The factored resistance of a brace shall be at least 2.0 per cent of the allowable compressive force in the member at the braced location.

14.7.2.2 Bracing by Deck, Slab or Sheathing

The requirements of Clause 20.3.2 shall be met.

14.7.3 Channels and Z-Sections Used in Flexure

14.7.3.1 Bracing shall be provided to resist twisting of channel and Z sections loaded in flexure in the plane of the web.

14.7.3.2 Bracing When Both Flanges are Braced by Deck or Sheathing Material

The allowable resistance of the attachment shall meet the requirements of Clause 20.3.2.

14.7.3.3 Bracing When One Flange is Braced by Deck or Sheathing Material

14.7.3.3.1 The allowable resistance of the attachment shall meet the requirements of Clause 20.3.2.

14.7.3.3.2 Discrete braces shall be provided to restrain the flange which is not braced by the deck slab or sheathing.

14.7.3.3.3 The spacing of discrete braces shall be in accordance with Clauses 14.7.3.4.1 and 14.7.3.4.2.

14.7.3.4 Bracing When Neither Flange is Braced by Deck or Sheathing Material

The following provisions for the spacing and design of discrete braces shall apply.

14.7.3.4.1 Spacing of Braces

Braces shall be attached both to the top and bottom flanges of the sections at the ends and at intervals not greater than one-quarter of the span length in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces, unless it can be shown by rational analysis or testing, taking into account both the effects of lateral and torsional displacements, that fewer braces can be used. If one-third or more of the total load on the beams is concentrated over a length of one-twelfth or less of the span, an additional brace shall be placed at or near the center of this loaded length.

14.7.3.4.2 Design of Flange Braces

Each intermediate brace, at the top and bottom flange, shall be designed to resist a lateral force, a function of the braced length, a, as follows:

(a) For a uniformly loaded beam

$B = 1.5K'aq$

(b) For concentrated loads when

$0 < x < 0.3a \quad P = K'P$

$0.3 < x < 1.0a$ $\qquad P = 1.43(1 - \frac{x}{a})K'P$

where

x = distance from concentrated load P to brace
K' = m/d for channels
K' = I_{xy}/I_x for Z-sections

Braces shall be designed to avoid local crippling at the points of attachment to the member.

End braces shall be designed for one-half the above forces.

14.7.3.5 Allowable Bending Stresses
The allowable bending stress for channels and Z-beams braced at intermediate points according to the requirements of Clauses 14.7.2.2 and 14.7.2.3, shall be determined in accordance with Clause 14.5.3, using, a, as the unbraced length.

14.7.4 Laterally Unbraced Box Beams
For closed box-type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed $17000/F_y$.

14.8 Testing

14.8.1 General
Test facilities shall be suitable for the type of test. Tests may be made at a manufacturer's or an independent testing facility. Test results and reports for type B and C tests shall be certified by a professional engineer. The provisions of this section do not apply to steel deck diaphragms and to composite steel components of composite steel assemblies.

14.8.2 Types of Tests
Tests are classified as follows:

Type A - Cold Formed Steel Properties
Full section tests to determine the modified mechanical properties of steel after cold working or cold forming for the utilization of the change in strength as permitted in Clause 14.4.1.2.

Type B - Performance Tests
Structural performance tests to establish the load carrying capacity of structural elements or assemblies for which the composition or configuration are such that calculation of the their safe-

load carrying capacity or deformation cannot be made in accordance with the provisions of this Standard.

Type C - Confirmatory Tests
Confirmatory tests to verify the safety factors of structural elements or assemblies designed in accordance with the provisions of this Standard. In no case shall the safety factors established by these tests be taken to be greater than that computed in accordance with the provisions of this Standard.

14.8.3 Test Procedures

14.8.3.1 Type A

14.8.3.1.1 Tensile testing procedures shall be performed in accordance with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370.

14.8.3.1.2 Comprehensive yield strength determinations shall be made by means of compression tests of short specimens of the section. The compressive yield strength shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross-section area or the strength defined by one of the following methods:

(i) for sharp yielding steel the yield strength shall be determined by the autographic diagram method or by the total strain under load method. When the total strain under load method is used, there shall be evidence that the yield strength so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method.

(ii) for gradual yielding steel the yield point shall be determined by the strain under load method or by the 0.2 percent offset method. When the total strain under load method is used, there shall be evidence that the yield strength so determined agrees within 5 percent with the yield strength which would be determined by the 0.2 percent offset method.

14.8.3.1.3 Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield strength shall be determined for the flanges only. In determining such yield strengths, tests shall be made on specimens cut from the section. Each such specimen shall consist of one complete flange plus a portion of the web such that the specimen is fully effective.

14.8.3.1.4 For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tonnes nor less than 30 tonnes of each section. For this purpose, lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat or blow.

14.8.3.1.5 At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield strength of the section when subjected to the type of stress under which the member is to be used.

14.8.3.2 **Type B**

14.8.3.2.1 Testing procedures shall be subject to the approval of the Regulatory Authority. Due considerations shall be given to the duration of load and boundary conditions in service of the elements and assemblies.

14.8.3.2.2 The allowable stresses shall be taken as the stress at failure divided by the appropriate factor of safety as given in Clause 14.5.1. If the critical property (yield strength, ultimate tensile strength, modulus of elasticity, etc.) of the steel from which the test sections are formed is larger than the specified value, the test results shall be adjusted to the specified minimum value of this property for the steel which the manufacturer intends to use. When the critical property is less than the minimum specified no such adjustment shall be made. Variations between the tested and specified values of the geometric properties shall also be taken into account.

14.8.3.2.3 The test value shall be established based on the mean values resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all tests does not exceed ± 10 percent. If such deviation from the mean exceeds 10 percent, at least three more tests of the same kind shall be made. The average of the three lowest values of all tests made shall then be regarded as the test value.

14.8.3.3 **Type C**

Confirmatory tests shall be performed and analyzed as for type B tests and shall meet the following criteria:

(a) the stress due to the specified loads shall not exceed the allowable stresses

(b) the deformations due to the specified loads shall not exceed the allowable values

14.9 Fabrication

14.9.1 Forming, Cutting, Punching and Drilling
Members shall be formed at ambient temperature by a method which does not result in work hardening to an extent that would limit the intended service and, where applicable, which does not result in damage to protective coatings which have been applied to the unformed material. Components may be cut by slitting, shearing, sawing, or flame cutting. Holes for fasteners may be punched or drilled.

14.9.2 Fastenings
Steel components may be assembled by means of welding or by the use of mechanical fasteners such as bolts, rivets or screws. Where dissimilar metals are fastened together attention shall be paid to electrical separation and the selection of suitable fasteners, recognizing the possibility of galvanic corrosion.

Fastenings such as metal stitching, clinching and structural adhesives also may be used where suitable. The strength of fastenings shall be established by test in accordance with Clause 14.8.3.2, unless values are specified elsewhere in this Standard.

14.9.3 Profiles and Distortion
Cold formed steel members shall be made to the full dimensions claimed by the manufacturer. Care shall be taken not to stretch, bend or otherwise distort parts of cold formed members except as a necessary feature of the cold forming operation.

14.10 Protection

14.10.1 Protection During Assembly, Storage, and Erection
Cold formed members shall be adequately protected from corrosion and deformation during assembly, storage and erection.

14.10.2 **Uncoated Stock**
All uncoated steel stock shall be stored in a dry place before processing and, except for weathering grades, shall be protected by a rust inhibitive coating immediately after processing.

14.10.3 **Coatings**
Cold formed members other than those made of weathering grades of steel, shall be protected against corrosion by means of paint, zinc, aluminum, porcelain enamel, or other effective means, either singly or in combination.

14.10.4 **Preparation of Surfaces for Coatings**
Before applying a protective coating, steel surface shall be dry, clean, and free from dirt, grease, loose or heavy scale and rust. When preparing welded assemblies for painting, the area at or near welds shall be thoroughly cleaned. After surfaces have been cleaned, a protective coating shall be applied as soon as practicable and before noticeable oxidation of clean surfaces occurs.

14.10.5 **Adequate Adhesion of Protective Coatings**
Careful consideration shall be given to the selection of a protective coating system to ensure that all procedures are compatible and will be such as to ensure adequate adhesion of the coating film.

15. Fatigue

15.1 General

15.1.1 In addition to meeting the requirements of Clause 15 for fatigue, any member or connection shall also meet the requirements for the static load conditions.

15.1.2 Members and connections subject to fatigue loading shall be designed, detailed and fabricated so as to minimize stress concentrations and abrupt changes in cross-section.

15.1.3 Specified loads for the design of members or connections shall be used for all fatigue calculations.

15.1.4 A specified load less than the maximum specified load but acting with a greater number of cycles may govern and shall be considered.

15.1.5 Plate girders with $h/w > 3150/\sqrt{F_y}$ shall not be used under fatigue conditions.

15.1.6 Slotted holes shall not be used in bolted connections in members subject to fatigue.

15.2 **Life**

For guidance in determining the number of cycles the life of the building should be assumed to be not less than 50 years unless otherwise stated.

15.3 **10 000 Cycles of Load**

When a load is expected to be applied not more than 10 000 times during the life of the structure, no special considerations beyond those in Clause 15.1.2 need apply.

15.4 **Over 10 000 Cycles of Load**

When a load is expected to be applied more than 10 000 times in the life of the structure, the loaded members, connections, bolts, and welds shall be proportioned so that the probability of fatigue failure is acceptably small. In such cases the design should be based on the best available information as to the fatigue characteristics of the materials and components to be used. In the absence of more specific information, Clause 15.5 provides guidance in proportioning members and parts. Fatigue resistance shall be provided only for those loads considered to be repetitive and hence contributing to fatigue. Often the magnitude of a repeated load is less than the maximum static load which the member or part would be designed to sustain.

15.5 **Allowable Range of Stress in Fatigue**

When this Clause is used as the basis for design, the members, connections, bolts and welds shall be proportioned so that the computed range of stress does not exceed the allowable range of stress F_{sr} given in Table 11(a) for the appropriate type and location of material shown in Table 11(b). The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress. Members subject to a range of stress involving only compression need not be designed for fatigue. The information in Tables 11(a) and 11(b) is shown diagramatically in Appendix J.

15.6 **Secondary Effects**

Secondary stresses, stresses due to deformations, and stresses due to out-of-plane movements are potential sources of fatigue cracks. Caution is

Table 11(a)
Allowable Ranges of Stress in Fatigue

Category (see Table 11(b) and Figure J2)	F_{sr} (MPa) For 100 000 Cycles	For 500 000 Cycles	For 2 000 000 Cycles	Over 2 000 000 Cycles
A	415	250	165	165
B	310	190	125	110
C	220	130	90	70*
D	185	110	70	48
E	145	85	55	32
F	110	65	40	18
W	115	85	65	48

* Except for transverse stiffener welds on girder webs or flanges, where 83 MPa may be used.

therefore advised in detailing structures which are subjected to repetitive loads and in which these sources of stresses may be present.

15.7 Single Load Paths

Structures in which the failure of a single element could result in collapse or catastrophic failure require special attention when fatigue cracking is a possibility. When Clauses 15.1 to 15.6 are followed in the design, the structure shall be subject to periodic inspection and maintenance. Alternatively, the permissible stress ranges shall be limited to 0.80 times those given in Table 11(a).

16. Beams and Girders

16.1 Proportioning

Beams and girders consisting of rolled shapes (with or without cover plates), hollow structural sections, or fabricated sections shall be proportioned on the basis of the properties of the gross section or the modified gross section as noted below. No deduction shall be made for fastener holes in webs or flanges unless the reduction of flange area by such holes exceeds 15 percent of the gross flange area, in which case the excess shall be deducted. The effect of openings other than holes for fasteners shall be considered in accordance with Clause 16.10.

Table 11(b)
Description of Design Conditions for Various Joint Classifications

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Plain Material	S1	Base metal with rolled or cleaned surfaces. Flame cut edges with a surface roughness not exceeding 1000 (25 μm) as defined by CSA Standard B95.	A	1, 2
Built-up Members	S2	Base metal and weld metal in members without attachments, built-up of plates or shapes connected by continuous complete or partial penetration groove welds or by continuous fillet welds, parallel to the direction of applied stress.	B	3,4,5
	S3	Base metal and weld metal along the length of horizontal stiffeners and cover plates connected by continuous complete or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	B	7
	S4	Base metal at toe of transverse stiffener welds on girder webs or flanges subjected to calculated flexural stress.	C	6
	S5	Base metal at end of longitudinal stiffeners.	E	7
	S6	Base metal at end of partial length welded cover plates narrower than the flange, having square or tapered ends, with or without welds across the ends.		
		Flange thickness < 20 mm	E	7
		Flange thickness > 20 mm	F	

Built-up members continued ...

Table 11(b) continued

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Built-up Members	S7	Base metal at end of partial length cover plates wider than the flange having square ends with welds across the ends.		
		Flange thickness ≤ 20 mm	E	7
		Flange thickness > 20 mm	F	
Complete Joint Penetration Grooves	S8	Base metal and weld metal at complete penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by non-destructive examination.*	B	8, 10
	S9	Base metal and weld metal in or adjacent to complete penetration groove welded splices at transition in width or thickness with welds ground to provide slopes no steeper than 4 in 10, with grinding in the direction of applied stress, and weld soundness established by non-destructive examination.* A 600 mm curved radius transition shall be used for CAN3-G40.21M-700Q and 700QT steel.	B	11, 12
	S10	Base metal and weld metal in or adjacent to complete penetration groove welded splices, involving equal widths and/or thicknesses of material, or involving transitions having slopes no greater than 4 in 10 when in either case reinforcement is not removed and when weld soundness is established by non-destructive examination.*	C	8,10,11,12

Complete Joint Penetration Groove continued ...

* For methods of non-destructive examination, see CSA Standard W59.

Table 11(b) continued

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Complete Joint Penetration Grooves	S11	For base metal at details attached to flanges or webs by groove welds subjected to transverse or longitudinal loading - regardless of detail length except for conditions as covered by Note (1) in tabulation for Example 13 the stress range categories shall be as shown in Fig. J2 in the tabulation for the sample example. Besides being dependent on transition radius, the stress range categories, in the case of flange connections subject to transverse loading, are also a function of relative thickness of material and whether or nor groove weld reinforcement is removed.	See Tabulation in Ex. 13 Fig. J2	13
Fillet Welded Connections	S12	Base metal at intermittent fillet welds.	E	
	S13	Base metal adjacent to fillet welded attachments where length L of the attachment in direction of stress is less than 50 mm.	C	6,14,15,16
	S14	Base metal at details attached by fillet welds subjected to longitudinal loading only when the detail length, L in direction of stress is between 50 mm and 12 times the plate thckness, but less than 100 mm and the transition radius R is less than 50 mm.	D	15

Fillet Welded Connections continued ...

Table 11(b) continued

General Condition	S-No.	Description	Stress Category (see Table 11(a))	Illustrative Example (see Figure J2)
Fillet Welded Connections	S15	For base metals at details attached to webs by fillet welds subjected to transverse and/or longitudinal loading regardless of detail length the stress range categories shall be as shown in Figure J2 in the tabulation for the sample Example. Shear stress on the throat of fillet welds shall be governed by stress range category "W".	See Tabulation in Ex. 13 Fig. J2	13
	S16	Except for cover plates (S6, S7) and details attached to webs (S15) base metal at end of details 100 mm or longer attached by fillet welds where the length of weld is in the direction of stress.	E	17
Fillet Welds	S17	Shear stress on throat of fillet welds.	W	17
Stud Type Shear Connections	S18	Shear stress on the nominal area of stud shear connectors.	W	
Mechanically Fastened Connections	S19	Base metal at gross section of high-strength bolted slip-resistant connections, except axially loaded joints which induce out-of-plane bending in connected material.	B	9
	S20	Base metal at net section of high-strength bolted bearing-type connections and other mechanically fastened joints.	B	9
	S21	Base metal at net section of bolted connections other than high-strength bearing-type.	D	9

16.2 **Rotational Restraint at Points of Support**

Beams and girders shall be restrained against rotation about their longitudinal axes at points of support.

16.3 **Reduced Allowable Bending Stress of Girders With Thin Webs**

When the web slenderness ratio, h/w, exceeds $1810/\sqrt{f_{bc}}$ the flange must meet the width-thickness ratios of Class 3 sections of Clause 11 and the allowable bending stress of the beam or girder, F'_{bc}, shall be determined by:

$$F'_{bc} = F_{bc}\left[1.0 - 0.0005\,\frac{A_w}{A_f}\left(\frac{h}{w} - 1810/\sqrt{f_{bc}}\right)\right]$$

where

F_{bc} = allowable bending stress determined by Clause 13.5 but not to exceed F_y.

16.4 **Flanges**

16.4.1 Flanges of welded girders preferably shall consist of a single plate or a series of plates joined end-to-end by complete penetration groove welds.

16.4.2 Flanges of bolted girders shall be proportioned so that the total cross-sectional area of cover plates does not exceed 70 per cent of the total flange area.

16.4.3 Fasteners or welds connecting flanges to webs shall be proportioned to resist horizontal shear forces due to bending combined with any loads which are transmitted from the flange to the web other than by direct bearing. Spacing of fasteners or intermittent welds in general shall be in proportion to the intensity of the shear force and shall not exceed the maximum for compression or tension members as applicable, in accordance with Clause 19.

16.4.4 Partial length flange cover plates shall be extended beyond the theoretical cut-off point and the extended portion shall be connected with sufficient fasteners or welds to develop a force in the cover plate at the theoretical cut-off point not less than:

$$P = \frac{AM_c y}{I_g}$$

where

P = required force to be developed in cover plate
A = area of cover plate
M_c = moment at point of theoretical cut-off
y = distance from centroid of cover plate to neutral

axis of cover-plated section
I_g = moment of inertia of cover-plated section

Additionally, for welded cover plates, the welds connecting the cover plate termination to the beam or girder shall be designed to develop the force P defined above within a length a' measured from the actual end of the cover plate, determined as follows:

(a) a' = the width of cover plate when there is a continuous weld equal to or larger than three-fourths of the cover plate thickness across the end of the plate and along both edges in the length a'

(b) a' = 1.5 times the width of cover plate when there is a continuous weld smaller than three-fourths of the cover plate thickness across the end of the plate and along both edges in the length a'

(c) a' = 2 times the width of cover plate when there is no weld across the end of the plate but continuous welds along both edges in the length a'

16.5 Bearing Stiffeners

16.5.1 Pairs of bearing stiffeners on the webs of single-web beams and girders shall be required at points of concentrated loads and reactions wherever the allowable bearing stress on the web is exceeded (see Clause 16.8). Bearing stiffeners shall be required also at unframed ends of single-web girders having web slenderness ratios greater than $1100/\sqrt{F_y}$. Box girders may employ diaphragms designed to act as bearing stiffeners.

16.5.2 Bearing stiffeners shall bear against the flange or flanges through which they receive their loads, and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns in accordance with Clause 13.3, assuming the column section to comprise the pair of stiffeners and a centrally located strip of the web equal to not more than 25 times its thickness at interior stiffeners, or a strip equal to not more than 12 times its thickness when the stiffeners are located at the end of the web. The effective column length, KL, shall be taken as not less than three-fourths of the length of the stiffeners in computing the ratio KL/r. Only that portion of the stiffeners outside of the angle fillet or the flange-to-web welds shall be considered effective in bearing. Angle bearing stiffeners shall not be crimped. Bearing stiffeners shall be connected to the web so as to develop the full force required to be carried by the stiffener into the web or vice versa.

16.6 Intermediate Transverse Stiffeners

16.6.1 Intermediate transverse stiffeners when used shall be spaced to suit the allowable shear stresses determined from the formula given in Clause 13.4; except that at girder end panels or at panels containing large openings, the smaller panel dimension, a or h, shall not exceed $890w/\sqrt{f_v}$ where f_v is the largest average shear stress in the panel.

16.6.2 The maximum distance between stiffeners, when stiffeners are required, shall not exceed the values shown in Table 12. Closer spacing may be required in accordance with Clause 16.6.1.

Table 12
Maximum Intermediate Transverse Stiffener Spacing

Web Slenderness Ratio (h/w)	Maximum Distance Between Stiffeners (a) in Terms of Clear Web Depth (h)
Up to 150	3h
Over 150	$\frac{67\ 500h}{(h/w)^2}$

16.6.3 Intermediate transverse stiffeners may be furnished singly or in pairs. Width-thickness ratios shall conform to Clause 11. The moment of inertia of the stiffener, or pair of stiffeners if so furnished, shall be not less than $(h/50)^4$ mm^4 taken about an axis in the plane of the web. The gross area of intermediate stiffeners, or pairs of stiffeners if so furnished, shall be given by the expression

$$A_s > \frac{aw}{2}\left[1 - \frac{a/h}{\sqrt{1 + (a/h)^2}}\right] CYD$$

where

a = distance centre-to-centre of adjacent stiffeners (i.e. panel length)

w = web thickness

h = web depth

$$C = 1 - \frac{310\ 000k_v}{F_y(h/w)^2}$$ but not less than 0.10

Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel

D = stiffener factor
= 1.0 for stiffeners furnished in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners

k_v = shear buckling coefficient (see Clause 13.4.1)

F_y = specified minimum yield point of web steel.

When the greatest stress, f_v, in an adjacent panel is less than that permitted by Clause 13.4.1, this gross area requirement may be reduced in like proportion by multiplying by the ratio f_v/F_v.

16.6.4 Intermediate transverse stiffeners shall be connected to the web for a shear transfer per pair of stiffeners (or per single stiffener when so furnished), in newtons per millimetre of web depth (h), not less than $6 \times 10^{-5} hF_y^{3/2}$; except that when the largest computed shear stress f_v in the adjacent panels is less than F_v computed by Clause 13.4.1 this shear transfer may be reduced in the same proportion. However the total shear transfer shall in no case be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener. Fasteners connecting intermediate transverse stiffeners to the web shall be spaced not more than 300 mm on centre. If intermittent fillet welds are used, the clear distance between welds shall not exceed 16 times the web thickness or 4 times the weld length.

16.6.5 When intermediate stiffeners are used on only one side of the web, the stiffeners shall be attached to the compression flange. Intermediate stiffeners used in pairs shall have at least a snug fit against the compression flange. When stiffeners are cut short of the tension flange the distance cut short shall be equal to or greater than 4 times but not greater than 6 times the girder web thickness. Stiffeners preferably shall be clipped to clear girder flange-to-web welds.

16.7 Lateral Forces

The flanges of beams and girders supporting cranes or other moving loads shall be proportioned to that the resulting stress distributions satisfy equilibrium.

16.11 **Torsion**

16.11.1 Beams and girders subjected to torsion shall have sufficient strength and rigidity to resist the torsional moment and forces in addition to other moment or forces. The connections and bracing of such members shall be adequate to transfer the reactions to the supports.

16.11.2 Torsional stresses, including those due to warping, shall be combined with stresses produced by plane bending and axial forces. The resulting combined stresses shall not exceed those prescribed by Clauses 13, and, if applicable, Clause 15.

16.11.3 For all members subjected to loads causing torsion, the torsional deformations under specified loads shall be limited in accordance with the requirements of Clause 6.2.1.1.

17. Open-Web Steel Joists

17.1 **Scope**
Clause 17 provides requirements for the design, manufacture, transport and erection of open-web steel joists used in construction of buildings. Joists intended to act compositely with the deck shall be designed using the requirements of Clause 18 in conjunction with the requirements of this Clause. This Clause shall not be used for the design of joists not having an axis of symmetry in the plane of the joist.

17.2 **Definitions**
The following definitions apply to Clause 17:

Open-web joists or joists means simply supported steel trusses of relatively low mass with parallel or slightly pitched chords and triangulated web systems proportioned to span between masonry walls, or structural supporting members, or both, and provide direct support for floor or roof deck.

Open-web steel joists are flexural members whose design is governed by the loading given in Clause 17.5.1. The definition does not include primary trusses supporting joists, other secondary members, and special joists.

Special open-web steel joists or special joists means:

(a) Joists subjected to the loads stipulated in Clause 17.5.2; and

(b) Cantilever joists, continuous joists and joists having special support conditions; and

(c) Joists having other special requirements.

In general open-web steel joists and special open-web steel joists are manufactured on a production-line basis employing jigs, certain details of the members being standardized by the individual manufacturer.

Deck or decking means the structural floor or roof element spanning between adjacent joists and directly supported thereby. The terms deck and decking include cast-in-place or precast concrete slabs, profiled metal deck, wood plank or plywood and other relatively rigid elements suitable for floor or roof construction;

Tie joist means a joist which has at least one end connected to a column to facilitate erection and is designed to resist gravity loads only unless otherwise specified;

Span of an open-web steel joists means the distance centre-to-centre of joists bearings.

17.3 Materials

Steel for joists shall be of a structural quality, suitable for welding, meeting the requirements of Clause 5.1.1. Yield levels reported on mill test certificates shall not be used as the basis for design.

17.4 Drawings

17.4.1 Building Design Drawings

The building design drawings prepared by the building designer shall show:

(a) The uniformly distributed specified live and dead gravity loads, the unbalanced loading condition and the concentrated load conditions given in Clause 17.5.1 or 17.5.2 and any special loading conditions such as horizontal loads, end moments, net uplift, and allowances for mechanical equipment;

(b) Maximum joist spacing and where necessary camber, maximum joist depth and shoe depth;

(c) Where joists are not supported on steel members, maximum bearing pressures or sizes of bearing plates;

(d) Anchorage requirements in excess of the requirements of Clause 17.5.13;

(e) Bracing as may be required by Clause 17.8.1.

Note: It is recommended that the building design drawings include a note warning that attachments for mechanical, electrical and other services shall be made by using approved clamping devices or u-bolt type connectors and that no drilling or cutting shall be done unless approved by the building designer.

17.4.2 **Joist Design Drawings**

Joist design drawings prepared by the joist manufacturer shall show, at least, specified loading, member loads, material specification, member sizes, dimensions, spacers, welds, shoes, anchorages, bearings, field splices, bridging locations and camber.

17.5 **Design**

17.5.1 **Loading for Open-Web Steel Joists**

Unless otherwise specified by the building designer (in accordance with Clause 17.5.2), the moment and shear capacity of an open-web steel joist at every section shall be not less than the moment and shear due to the following load conditions, considered separately:

(a) A uniformly distributed load equal to the total dead and live load;

(b) An unbalanced load with 100 per cent of the total dead and live load on any continuous portion of the joist and 25 per cent of total dead and live loads on the remainder to produce the most critical effect on any component;

(c) A concentrated factored load applied at any panel point of 9 kN for floor joists for office or similar occupancy or 1.3 kN for roof joists.

17.5.2 **Loading for Special Open-Web Steel Joists**

The moment and shear capacities of special open-web steel joists at every section shall be not less than the moment and shear due to the loading conditions specified by the building designer in Clause 17.4.1(a) nor due to the dead load plus the following live load conditions (a), or (b) considered separately:

(a) For floor joists, an unbalanced live load applied on any continuous portion of the joist to produce the most critical effect on any component;

(b) The appropriate concentrated load specified by the Regulatory Authority; applied at any one panel point to produce the most critical effect on any component.

17.5.3 **Design Assumptions**

Open-web steel joists shall be designed for loads acting in the plane of the joist applied to the compression chord which is assumed to be prevented from lateral buckling by the deck.

For the purpose of determining axial forces in all members the loads may be replaced by statically equivalent loads applied at the panel points.

17.5.4 **Verification of Joist Manufacturer's Design**

When the adequacy of the design of a joist cannot be readily demonstrated by a rational analysis based on accepted theory and engineering practice, the joist manufacturer may elect to verify the design by test. The test shall be carried out to the satisfaction of the building designer. The test loading shall be 1.10 times the applicable factor of safety times the design load.

17.5.5 **Member and Connection Capacity**

Member and connection capacity shall be calculated in accordance with the requirements of Clause 13 except as otherwise specified in Clause 17.

17.5.6 **Width-Thickness Ratios**

17.5.6.1 **General**

Width-thickness ratios of compression elements of hot formed sections shall be governed by Clause 11. Width thickness ratios of compression elements of cold formed sections shall be governed by Clause 14.

17.5.6.2 **Compression Elements Supported Along One Edge**

For purposes of determining the appropriate width-thickness ratio, any stiffening effect of the deck or the joist web shall be neglected.

17.5.7 **Tension Chord**

The tension chord shall be continuous and may be designed as an axially loaded tension member unless subject to eccentricities in excess of those permitted under Clause 17.5.11.4 or to applied load between panel points. The governing radius of gyration of the tension chord or any component thereof shall be not less than 1/240 of the corresponding unsupported length. For joists with the web in the y-plane the unsupported length of chord for computing L_x/r_x shall be taken as the panel length centre-to-centre of panel points and the unsupported length of chord for computing L_y/r_y shall be taken as the distance between bridging lines connected to the tension chord. Joist shoes, when anchored, may be assumed to be equivalent to bridging lines. When net uplift is specified, the tension chord shall be designed for the resulting load reversal. Where shown on the drawings, bottom chords of joists shall be designed for end moments. Moments due to concentrated loads shall be included in the design.

17.5.8 Compression Chord

17.5.8.1 The compression chord shall be continuous and may be designed for axial compressive force alone when the panel length does not exceed 610 mm, when concentrated loads are not applied between the panel points, and when not subjected to eccentricities in excess of those permitted under Clause 17.5.11.4. When the panel length exceeds 610 mm the compression chord shall be designed as a continuous member subject to combined axial and bending forces.

17.5.8.2 The slenderness ratio (KL/r) of the compression chord, or of its components, shall not exceed 90 for interior panels nor 120 for end panels where the governing (KL/r) shall be the maximum value determined by the following:

(a) For x-x (horizontal) axis, L_x shall be the distance centre-to-centre of panel points. K = 0.9;

(b) For y-y (vertical) axis, L_y shall be the distance centre-to-centre of the attachments of the deck. The spacing of attachments shall be not more than the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis nor more than 1000 mm. K = 1.0;

(c) For z-z (skew) axis of individual components, L_z shall be the distance centre-to-centre of panel points or spacers, or both. Decking shall not be considered to fulfil the function of batten plates or spacers for top chords consisting of two separated components. K = 0.9.

where

r = the appropriate radius of gyration.

17.5.8.3 Compression chords of joists in panel lengths exceeding 610 mm shall be proportioned such that:

(a) At panel points

$$\frac{f_a}{0.60F_y} + \frac{f_b}{F_b} < 1.0$$

(b) At mid-panel

$$\frac{f_a}{F_a} + \frac{f_a}{F_b} < 1.0$$

where

F_a = permissible axial compressive stress (see Clause 17.5.5)

F_b = permissible bending stress (see Clause 17.5.5)

At the panel point F_b may be taken as $0.75F_y$ provided that the chord meets the requirements of plastic design or compact sections of Clause 11.2 and $f_b/F_b < 0.25$.

The chord shall be assumed to be pinned at the joist supports.

17.5.9 Webs

17.5.9.1 Webs shall be designed in accordance with the requirements of Clause 13 to resist the shear at any point due to the loads given in Clause 17.5.1 or 17.5.2. Particular attention shall be paid to possible reversals of shear.

17.5.9.2 The length of a web member shall be taken as the distance between the intersections of the axes of the web and the chords. For buckling in the plane of the web the effective length factor shall be taken as 0.9 if the web consists of individual members. For all other cases the effective length factor shall be taken as 1.0.

17.5.9.3 **Web Members in Tension**
The slenderness ratio of a web member in tension need not be limited.

17.5.9.4 **Web Members in Compression**
The slenderness ratio of a web member in compression shall not exceed 200.

17.5.10 **Spacers and Battens**
Compression members, consisting of two or more sections, shall be interconnected so that the slenderness ratio of each section computed using its least radius of gyration is less than or equal to the design slenderness ratio of the built-up member. Spacers or battens shall be an integral part of the joist.

17.5.11 **Connections and Splices**

17.5.11.1 Component members of joists shall be connected by welding, bolting or other approved means.

17.5.11.2 Connections and splices shall develop the loads required by this Standard without exceeding the stresses given in Clause 17. Groove-welded splices shall develop the tensile strength of the member.

17.5.11.3 Splices may occur at any point in chord or web members.

17.5.11.4 **Eccentricity Limits**
Members connected at a joint preferably shall have their gravity axes meet at a point. Where this is impractical and eccentricities are introduced such eccentricities may be neglected if they do not exceed:

(a) **For continuous web members** - The greater of the two distances measured from the neutral axis of the chord member to the extreme fibres of the chord member.

(b) **For non-continuous web members** - The distance measured from the neutral axis to the back (outside face) of the chord member.

When the eccentricity exceeds these limits, provision shall be made for the effects of total eccentricity.

Eccentricities assumed in design shall be those at maximum fabrication tolerances which shall be stated on the shop drawings.

17.5.12 **Bearings**

17.5.12.1 Bearings at ends of joists shall be proportioned so that the factored bearing resistance of the supporting material is not exceeded.

17.5.12.2 Where a joist bears, with or without a bearing plate on solid masonry or concrete support, the end of the bearing shall extend at least 90 mm beyond the face of support.

17.5.12.3 Where a joist bears on a member of the structural steel frame, the end of the bearing shall extend at least 65 mm beyond the face of the support except that when the available bearing area is restricted, this distance may be reduced provided that the bearing is adequately anchored to the support and the bearing capacity is not exceeded.

17.5.12.4 The bearing detail and the end panels of the joist shall be proportioned to include the effect of the eccentricity between the centre of bearing and the intersection of the axes of the chord and the end diagonal.

17.5.13 Anchorage

17.5.13.1 Joist ends shall be properly anchored to withstand the effect of applied loads:

(a) In no case shall the anchorage to masonry be less than:

(i) For floor joists, a 10 mm diameter rod at least 300 mm long embedded horizontally;

(ii) For roof joists, a 20 mm diameter anchor bolt 300 mm long embedded vertically with a 50 mm - 90° hook;

(b) The anchorage to steel shall be a connection capable of withstanding a horizontal load not less than 10 percent of the end reaction of the joist but not less than one 20 mm diameter bolt or a pair of fillet welds satisfying the minimum size and length requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

17.5.13.2 **Tie Joists**

Tie joists may have their top and bottom chords connected to a column. Unless otherwise specified, tie joists shall have top and bottom chord connections each at least equivalent to those required by Clause 17.5.13.1. Either top or bottom connection shall be by means of a mechanical fastener.

17.5.13.3 **Frame Action**
Where joists are used as a part of a frame, the joist to column connection shall be designed to carry the moments and forces due to the applied loads (see Clause 7.2).

17.5.14 **Deflection**

17.5.14.1 **General**
Steel joists shall be proportioned so that deflection due to specified loads is within acceptable limits for the nature of the materials to be supported and the intended use and occupancy. Such deflection limits shall be as given in Clause 6.2 unless otherwise specified by the building designer.

17.5.14.2 **Deflection Calculations**
The deflection may be established by test or may be computed assuming a moment of inertia equal to the gross moment of inertia of the chords about the centroidal axis of the joist and multiplying the calculated deflection derived on this basis by 1.10.

17.5.15 **Camber**
Unless otherwise specified by the building designer the nominal camber in millimetres shall be equal to 0.07 times the square of the span expressed in metres. For tolerances see Clause 17.10.9.

17.5.16 **Vibration**
The building designer shall give special consideration to floor systems where unacceptable vibration may occur. The joist manufacturer when requested shall supply joist properties and details to the building designer. (See Appendix F.)

17.5.17 **Welding**

17.5.17.1 **Arc Welding**
Arc welding design and practice shall conform to CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

17.5.17.2 **Resistance Welding**
Resistance welding design and practice shall conform to the applicable requirements of CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Use in Buildings, and the related welding practice shall be in conformance with welding standards approved by the Canadian Welding Bureau under the same CSA Standard.

17.5.17.3 **Fabricator and Erector Qualification**
Fabricators and erectors of welded construction covered by this Standard shall be certified by the Canadian Welding Bureau in Division 1 or 2.1 to the requirements of CSA Standard W47.1, Certification of Companies for Fusion Welding of Steel Structures, or CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, or both, as applicable.

17.5.17.4 The permissible stresses for welds shall be those given in Table 5(b).

17.5.17.5 **Field Welding**
To achieve an adequate weldment when field welding joists to supporting members, surfaces to be welded shall be free of detrimental coatings.

17.5.17.6 **Removal of Flux and Slag**
Flux and slag shall be removed from all welds.

17.6 **Stability During Construction**
Means shall be provided to support joist chords against lateral movement and to hold the joist in the vertical or specified plane during construction.

17.7 **Bridging**

17.7.1 **General**
Bridging transverse to the span of joists may be used to meet the requirements of Clause 17.6 and also to meet the slenderness ratio requirements for chords. Bridging is not to be considered "bracing" as defined under Clause 20.3.1.

17.7.2 **Installation**
All bridging and bridging anchors shall be completely installed before any construction loads are placed on the joists except for the weight of the workmen necessary to install the bridging.

17.7.3 **Types**
Unless otherwise specified or approved by the building designer the joist manufacturer shall supply bridging which may be either the diagonal or horizontal type.

17.7.4 **Diagonal Bridging**
Diagonal bridging consisting of crossed members running from top chord to bottom chord of adjacent joists shall have a slenderness ratio (L/r) of not more than 200 where "L" is the length of the diagonal bridging member, or one-half this length when crossed members are connected at their point of

intersection, and "r" is the least radius of gyration. All diagonal bridging shall be connected adequately to the joists by bolts or welds.

17.7.5 **Horizontal Bridging**
A line of horizontal bridging shall consist of a continuous member attached to either the top chord or the bottom chord. Horizontal bridging members shall have a slenderness ratio of not more than 300.

17.7.6 **Attachment of Bridging**
Attachment of diagonal and horizontal bridging to joist chords shall be by welding or mechanical means capable of resisting an axial load of at least 2 kN in the attached bridging member. These welds should meet the minimum length requirements stipulated in CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

17.7.7 **Anchorage of Bridging**
Each line of bridging shall be adequately anchored at each end to sturdy walls or to main components of the structural frame, if practicable. If not practicable, diagonal and horizontal bridging shall be provided in combination between adjacent joists near the ends of bridging lines.

The ends of joists designed to bear on their bottom chords shall be held adequately in position by attachments to the walls or to the structural frame or by lines of bridging located at the ends except where such ends are built into masonry or concrete walls.

17.7.8 **Bridging Systems**
Bridging systems, including sizes of bridging members, and all necessary details, shall be shown on the erection diagrams. If a specific bridging system is required by the design, the design drawings shall show all information necessary for the preparation of shop details and erection diagrams.

17.7.9 **Spacing of Bridging**
Diagonal and horizontal bridging, whichever is furnished, shall be spaced so that the unsupported length of the chord between bridging lines, or between laterally supported ends of the joist and adjacent bridging lines, does not exceed:

(a) For compression chords, 170r;

(b) For tension chords, 240r;

where

r = the applicable chord radius of gyration about its axis in the plane of the web

Ends of joists anchored to supports may be assumed to be equivalent to bridging lines. If not so anchored before installing deck, the distance from the face of the support to the nearest bridging member in the plane of the bottom chord shall not exceed 120r. In no case shall there be less than one line of horizontal or diagonal bridging attached to each joist spanning 4000 mm or more. If only a single line of bridging is required, it shall be placed at the centre of the joist span. If bridging is not used on joists less than 4000 mm in span, the ends of such joists shall be anchored to the supports so as to prevent overturning of the joist during placement of the deck.

17.8 Decking

17.8.1 Decking to Provide Lateral Support

Decking shall bear directly on the top chord of the joist and shall be sufficiently rigid to provide lateral support to the compression chord of the joist. In special cases where the decking is incapable of furnishing the required lateral support, the compression chord of the joist shall be braced laterally in accordance with the requirements of Clause 20.3.

17.8.2 Attachments of decking considered to provide lateral support shall be capable of staying the top chords laterally. Attachments shall be deemed to fulfil this requirement when the attachments as a whole are adequate to resist a force in the plane of the decking of not less than 5 per cent of the maximum force in the top chord and assumed to be uniformly distributed along the length of the top chord. The spacing of attachments shall be not more than the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis nor more than 1000 mm.

17.8.3 Diaphragm Action

Where decking is used in combination with joists to form a diaphragm for the purpose of transferring lateral applied loads to vertical bracing systems, special attachment requirements shall be fully specified on the building design drawings.

17.8.4 Cast-in-place slabs used as decking shall have a minimum thickness of 50 mm. Forms for cast-in-place slabs shall not cause lateral displacement of the top chords of joists during installation of the forms or the placing of the concrete. Non-removable forms shall be positively attached to top chords by means of clips, ties, wedges, fasteners, or other suitable means at intervals not exceeding 1000 mm; however, there shall be at least two attachments in the width of each form at each joist. Forms and their method of attachment shall be such that the cast-in-place slab, after hardening, is capable of furnishing lateral support to the joist chords.

17.9 Shop Painting

Joists shall have one shop coat of protective paint of a type standard with the manufacturer unless otherwise specified.

17.10 Manufacturing Tolerances

17.10.1 The tolerance on the specified depth of the manufactured joist shall be ± 7 mm.

17.10.2 The maximum deviation from the design location of a panel point measured along the length of a chord shall be 13 mm. In joists in which an individual end diagonal is attached to the bottom chord or in which the end diagonal is a continuation of an upturned bottom chord the gravity axes of the members in such a joint should meet at a point. (See Clause 17.5.11.4.)

17.10.3 The maximum deviation from the design location of a panel point measured perpendicular to the longitudinal axis of the chord and in the plane of the joist shall be 7 mm.

17.10.4 The connections of web members to chords shall not deviate laterally more than 3 mm from that assumed in the design.

17.10.5 The maximum sweep of a joist or any portion of the length of the joist upon completion of manufacture shall be 1/500 of the length on which the sweep is measured.

17.10.6 The maximum tilt of bearing shoes shall be 1 in 50 measured from a plane perpendicular to the plane of the web and parallel to the longitudinal axis of the joist.

17.10.7 The tolerance on the specified shoe depth shall be ± 3 mm.

17.10.8 The tolerance on the specified length of the joist shall be ± 7 mm. The connection holes in a joist shall not vary from the detailed location by more than 2 mm for members 10 000 mm or less in length or by more than 3 mm for members over 10 000 mm in length.

17.10.9 The tolerance on the nominal or specified camber shall be

$$\pm\left(6\ \text{mm} + \frac{\text{Span, in mm}}{4\ 000}\right)$$

The resulting actual minimum camber in a joist is to be +3 mm except that the maximum range in camber for joists of the same span shall be limited to 20 mm.

17.11 Inspection and Quality Control

17.11.1 Inspection

Material and workmanship at all times shall be accessible for inspection by qualified inspectors representing the building designer. Random in-process inspection shall be carried out by the manufacturer and all joists shall be thoroughly inspected by the manufacturer before shipping.

17.11.2 Identification and Control of Steel

Steel used in the manufacture of joists shall at all times, in the manufacturers' plant be marked to identify its specification (and grade, where applicable). This shall be done by suitable markings or by recognized colour coding or by any system devised by the manufacturer that will ensure to the satisfaction of the building designer that the correct material is being used.

17.11.3 Quality Control

Upon request of the building designer the manufacturer shall provide evidence of having suitable quality control measures to ensure that the joists meet all requirements specified. When testing is part of the manufacturer's normal quality control program, the loading criterion shall be 1.67 times the design load.

For resistance welding and quality control procedures outlined in CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, shall be met. For arc-welding quality control, the requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding), shall be met.

17.12 Handling and Erection

17.12.1 General

Care shall be exercised to avoid damage during strapping, transport, unloading, site storage and piling, and erection. Dropping of joists shall not be permitted. Special precautions shall be taken when erecting long, slender joists and preferably hoisting cables shall not be released until the member is stayed laterally by at least one line of bridging. Joists shall have all bridging attached and be permanently fastened into place before the application of any loads. Heavy construction loads shall be adequately distributed so as not to exceed the capacity of any joist. Field welding shall not cause damage to joists, bridging, deck and supporting steel members.

17.12.2 Erection Tolerances

17.12.2.1 The maximum sweep of a joist or a portion of the length of a joist upon completion of erection shall not exceed the requirements of Clause 17.10.5, and shall be in accordance with the general requirements of Clause 29.

17.12.2.2 All members shall be free from twists, sharp kinks and bends.

17.12.2.3 Location of Joist

When joists are finally fastened in position in the field, the maximum deviation from the location shown on the erection drawings shall be 15 mm.

17.12.2.4 The deviation, normal to the specified plane of the web of a joist, shall not exceed 1/50 of the depth of the joist.

18. Composite Beams and Columns

18.1 Application

18.1.1 The provisions of Clause 18 apply to:

(a) Composite beams consisting of steel sections, trusses or joists interconnected with either a reinforced concrete slab or a steel deck with concrete cover slab;

(b) Composite columns consisting of steel hollow structural sections completely filled with concrete.

18.1.2 For any requirement not covered in Clause 18 the design shall conform to the provisions of this Standard.

18.2 **Definitions**

The following definitions apply to Clause 18.

Steel deck means a load-carrying steel deck consisting of either:

(a) A single fluted element (non-cellular deck) ; or

(b) A two element section comprising a fluted element in conjunction with a flat sheet (cellular deck).

The maximum depth of the deck shall be 80 mm and the average width of the minimum flute shall be 50 mm. Steel deck may be of a type intended to act compositely with the cover slab in supporting applied load;

Flute means that portion of the steel deck which forms a valley.

Concrete means Portland cement concrete in accordance with CSA Standard CAN3-A23.1, Concrete Materials and Methods of Concrete Construction;

Rib means that portion of the concrete slab which is formed by the steel deck flute.

Slab means a reinforced cast-in-place concrete slab at least 65 mm in effective thickness. The area equal to the design effective width times effective slab thickness shall be free of voids or hollows except for those specifically permitted in the definition of effective slab thickness;

Cover slab means the concrete above the flutes of steel deck. All flutes shall be filled with concrete so as to form a rib slab;

Effective cover slab thickness, t, means the minimum thickness of concrete measured from the top of the cover slab to the top of the steel deck. This thickness shall be not less than 65 mm unless the adequacy of a lesser thickness has been established by appropriate tests;

Effective slab thickness, t, means the overall slab thickness, provided that;

(a) The slab is cast with a flat underside; or

(b) The slab is cast on corrugated steel forms having a height of corrugation not greater than 0.25 times the overall slab thickness; or

(c) The slab is cast on fluted steel forms whose profile meets the following requirements. The minimum concrete rib width shall be 125 mm; the maximum rib height shall be 40 mm but not more than 0.4 times the overall slab thickness; the average width between ribs shall not exceed 0.25 times the overall slab thickness nor 0.2 times the minimum width of concrete.

In all other cases, effective slab thickness means the overall slab thickness minus the height of form flute or corrugation;

Steel joist means an open web steel joist suitable for composite design;

Steel section means a steel structural section with a solid web, or webs, suitable for composite design. Web openings are permissible only on condition that their effects are fully investigated and accounted for in the design.

18.3 **Composite Beams**

18.3.1 **General**

18.3.1.1 Calculation of deflections shall take into account the effects of creep of concrete, shrinkage of concrete, and increased flexibility resulting from partial shear connection and from interfacial slip. These effects shall be established by test or analysis, where practicable. Consideration shall also be given to effects of full or partial continuity in the steel beams and concrete slabs in reducing calculated deflections.

In lieu of tests or analyses the effects of partial shear connection, interfacial slip, creep and shrinkage may be assessed as follows:

(a) increased flexibility resulting from partial shear connection and interfacial slip: calculate the elastic deflections using an effective moment of inertia,

$$I_e = I_s + 0.85\, p^{0.25} (I_t - I_s)$$

where

I_s = moment of inertia of the steel section

I_t = moment of inertia of the transformed composite section

p = fraction of full shear connection expressed as a decimal

(b) creep: increase elastic deflections due to dead loads and long term live loads, as computed in (a), by 15%.

(c) shrinkage of concrete: calculate deflections using a selected free shrinkage strain assuming the beam is bent in single curvature by a constant moment. The shrinkage strain is affected by such factors as age of concrete, ratio of slab volume to surface area, and concrete properties (water/cement ratio, percent fines, entrained air, and cement content), and the restraint provided by steel beam and deck. See Appendix K for methods of computing deflections due to shrinkage strains.

18.3.1.2 The web area of steel sections or web system of steel trusses and joists shall be proportioned to carry the total vertical shear V_f.

18.3.1.3 End connections of steel sections, trusses and joists shall be proportioned to transmit the total end reaction of the composite beam.

18.3.2 Design Effective Width of Concrete

18.3.2.1 Slabs or cover slabs extending on both sides of the steel section or joist shall be deemed to have a design effective width, b, equal to the least of:

(a) 0.25 times the composite beam span;

(b) 16 times the overall slab thickness (thickness of cover slab plus steel deck depth), plus the width of the top flange of the steel section or top chord of the steel truss or joist;

(c) The average distance from the centre of the steel section, truss or joist to the centres of adjacent parallel supports.

18.3.2.2 Slabs or cover slabs extending on one side only of the supporting section or joist shall be deemed to have a design effective width, b, not greater than the width of top flange of the steel section, or top chord of the steel joist, plus the least of:

(a) 0.1 times the composite beam span;

(b) 6 times the overall slab thickness or (thickness of cover slab plus steel deck depth);

(c) 0.5 times the clear distance between the steel section or joist and the adjacent parallel support.

18.3.3 **Slab Reinforcement**

Slabs shall be adequately reinforced to support all specified loads and to control cracking both parallel and transverse to the composite beam span. Reinforcement parallel to the span of the beam in regions of negative bending moment of the composite beam shall be anchored by embedment in concrete which is in compression. The reinforcement of slabs which are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention.

The possibility of longitudinal cracking due to composite action, directly over the steel section or joist, shall be controlled by the provision of additional transverse reinforcement or other effective means unless it is known from experience that cracking due to composite action is unlikely. Such additional reinforcement shall be placed in the lower part of the slab and anchored so as to develop the yield strength of the reinforcement. The area of such reinforcement shall be not less than 0.005 times the concrete area in the longitudinal direction of the beam and shall be uniformly spaced along the composite beam span.

18.3.4 **Composite Action With Steel Deck**

Cover slabs intended to act compositely with steel deck shall have reinforcement transverse to the span of the composite beam as required. Reinforcement shall be not less than that required by the specified fire resistance design of the assembly.

18.3.5 **Interconnection**

18.3.5.1 Except as permitted by Clauses 18.3.5.2 and 18.3.5.4 interconnection between steel sections, trusses or joists and slabs or cellular steel deck with cover slabs shall be attained by the use of shear connectors as prescribed in Clause 18.3.6.

18.3.5.2 Unpainted steel sections, trusses or joists supporting slabs and totally encased in concrete do not require interconnection by means of shear connectors provided that:

(a) A minimum of 50 mm of concrete covers all portions of the steel section or joist, except as noted in Item (c);

(b) The cover in Item (a) is reinforced to prevent spalling; and

(c) The top of the steel section or joist is at least 40 mm below the top and 50 mm above the bottom of the slab.

18.3.5.3 Studs may be welded through a maximum of two steel sheets in contact, each not more than 1.71 mm in overall thickness including coatings (1.52 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise holes for placing studs shall be made through the sheets as necessary. Welded studs shall meet the requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

18.3.5.4 Other methods of interconnection which have been adequately demonstrated by test and verified by analysis may be used to effect the transfer of forces between the steel section, truss or joist and the slab or steel deck with cover slab. In such cases the design of the composite member shall conform to the design of a similar member employing shear connectors, insofar as practicable.

18.3.5.5 The diameter of a welded stud shall not exceed 2.5 times the thickness of the part to which it is welded, unless test data satisfactory to the designer is provided to establish the capacity of the stud as a shear connector.

18.3.6 **Shear Connectors**

The capacity, q, of a shear connector shall be established by tests acceptable to the designer, except that the following values shall be acceptable without further verification:

(a) **End welded studs, headed or hooked with h/d > 4**

$$q = 0.5A_{sc}\sqrt{f'_c E_c} < F_u{}^* A_{sc} \quad \text{(newtons)}$$

$^*F_u$ for commonly available studs is 415 MPa.

This value is limited to designs incorporating a solid concrete slab; or designs incorporating a ribbed slab formed by casting a concrete cover slab on a fluted steel deck in which the rib average width is at least twice the height of the formed concrete rib and the projection of the stud, based on its length prior to welding, is at least two stud diameters above the top surface of the steel deck;

(b) **End welded studs, headed, in selected cases -**

Table 13 gives values of q_r for selected cases of composite beams incorporating steel deck not covered by Item (a). Cover slabs shall consist of normal density concrete (2300 kg/m^3) with $f'_c \geq 20$ MPa.

(c) **Channel connectors**

$$q = 36.5\left(t_f + 0.5t_w\right) L_c\sqrt{f'_c} \quad \text{(newtons)}$$

This formula is limited to design incorporating a solid concrete slab of normal density concrete (2300 kg/m^3) with $f'_c \geq 20$ MPa.

Table 13
Shear Capacities of Studs for Selected Cases

Height of Cellular Steel Deck (mm)	Average Rib Width, Minimum (mm)	Depth of Cover Slab (mm)	Stud Size d x h (mm x mm)	No. of Studs per Rib	Shear Capacity* q (newtons)
38/43	50	65	14 x 75	1	24 000
38/43	50	65	20 x 75	1	50 000
				2	76 000 per pair
38/43	50	90	20 x 100	1	79 000
				2	113 000 per pair

* Shear capacities given in Table 13 are derived from test and reflect the influence of rib geometry and stiffness on the useful capacity of the studs.

18.3.7 **Ties**
Mechanical ties shall be provided between the steel section, truss or joist and the slab or steel deck to prevent separation. Shear connectors may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1000 mm and the average spacing in a span should not exceed 600 mm nor be greater than that required to achieve any specified fire resistance rating of the composite assembly.

18.4 Design of Composite Beams With Shear Connectors

18.4.1 The composite beam shall consist of steel section, truss or joist, shear connectors, ties, and slab or steel deck with cover slab.

18.4.2 The properties of the composite section shall be computed according to elastic theory neglecting any concrete area in tension within the maximum effective area equal to effective width times effective thickness. If a steel truss or joist is used the area of its top chord shall be neglected in determining the properties of composite section. The effective area of concrete in compression shall be considered as an equivalent area of steel by dividing it by the appropriate modular ratio.

18.4.3 The composite section shall be proportioned to resist the total specified load without exceeding the allowable stresses for steel and concrete respectively.

18.4.4 If fewer than the number of shear connectors required for full composite action are to be provided (see Clause 18.4.6) the effective area of concrete in compression to be assumed in design shall be reduced from that given in Clause 18.2 and 18.3 in proportion to the ratio n'/n.

where

n' = number of shear connectors provided
n =number of shear connectors required for full composite action according to Clause 18.4.6.

18.4.5 No composite action shall be assumed in computing flexural strength when $n' < 0.5n$.

No composite action shall be assumed in computing deflections when $n' < 0.25n$.

18.4.6 For full shear connection, the total horizontal shear, V_h, at the junction of the steel section or joist and the concrete slab or steel deck, to be resisted by shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment, shall be the lesser of

$$V_h = A_s F_y$$

$$V_h = 0.85btf'_c$$

18.4.7 Composite beams employing steel sections and concrete slabs may be designed as continuous members. The moment capacity of the composite section with the concrete slab in the tension area of the composite section shall be the moment resistance of the steel section alone except that when sufficient shear connectors are placed in the negative moment region, suitably anchored concrete slab reinforcement parallel to the steel sections and within the design effective width of the concrete slab may be included in computing the properties of the composite section. The total horizontal shear, V_h, to be resisted by shear connectors between the point of maximum negative bending moment and the adjacent point of zero moment shall be taken as:

$$V_h = A_r F_{yr}$$

18.4.8 The number of shear connectors to be located each side of the point of maximum bending moment (positive or negative, as applicable) and distributed between that point and the adjacent point of zero moment shall be not less than:

$$n = \frac{V_h}{q}$$

Shear connectors may be spaced uniformly except that in a region of positive bending, the number of shear connectors required between any concentrated load applied in that region and the nearest point of zero moment shall be not less than n"

$$n'' = n \left(\frac{M - M_s}{M_{max} - M_s}\right)$$

where

M = positive bending moment at concentrated load point
M_s = moment capacity of the steel section alone
M_{max} = maximum positive bending moment

18.5 Design of Composite Beams Without Shear Connectors

18.5.1 Unpainted steel sections or joists supporting concrete slabs and encased in concrete in accordance with Clause 18.3.5.2 may be proportioned on the basis that the composite section supports the total load.

18.5.2 In computing the moment of inertia of the composite section any concrete area in tension shall be neglected.

18.5.3 The maximum bending stress in the steel section or joist shall not exceed the allowable bending stresses given in Clause 13.5 for the class of section used and the maximum compressive bending stress in the concrete shall not exceed $0.45f'_c$.

18.6 Unshored Beams

For composite beams unshored during construction, the stresses in the tension flange of the steel section or joist due to the loads applied before the concrete strength reaches $0.75f'_c$ plus the stresses at the same location, due to the remaining specified loads considered to act on the composite section shall not exceed $0.90F_y$.

18.7 **Beams During Construction**

The steel section or joist alone shall be proportioned to support all loads applied prior to hardening of the concrete without exceeding its calculated capacity under the conditions of lateral support and shoring, as applicable, to be furnished during construction.

18.8 **Design of Composite Columns (Concrete-Filled Hollow Structural Sections)**

18.8.1 Hollow structural sections designated as Class 1, 2 or 3 sections which are completely filled with concrete may be assumed to carry compressive load as composite columns. Class 4 hollow structural sections completely filled with concrete may also be designed as composite columns providing the width-thickness ratios of the walls of rectangular sections do not exceed $1350/\sqrt{F_y}$, and the outside diameter to thickness ratio of circular sections do not exceed $28\,000/F_y$.

18.8.2 The proportion of the axial load assumed to be carried by the concrete shall be applied by direct bearing on the concrete, or, alternatively, other methods of load application to the concrete may be employed if their adequacy has been demonstrated by test.

18.8.3 The allowable axial compressive load of a composite column shall be taken as:

$$P_a = \tau P_s + \tau' P_c$$

where

P_s = allowable axial load on steel section alone

$= A_s F_a$ with F_a defined in Clause 13.3

P_c = allowable axial load on concrete area A_c

$$P_c = 0.35\, f'_c A_c \lambda^{-2}\left[\sqrt{1 + 0.25\lambda_c^{-4}} - 0.5\lambda_c^{-2}\right]$$

in which $\lambda_c = \dfrac{KL}{r_c}\sqrt{\dfrac{f'_c}{\pi^2 E_c}}$

r_c = radius of gyration of the concrete area, A_c,

E_c = initial elastic modulus for concrete in MPa, considering the effects of long term loading. For normal weight concrete, with f'_c expressed in MPa, this may be taken as:

$$(1 + S/T)\ 2500 \sqrt{f'_c}$$

where S is the short term load and T is the total load on the column.

For all rectangular hollow structural sections, and for circular hollow structural sections with height to diameter ratio of 25 or greater, $\tau = \tau' = 1.0$

Otherwise $$\tau = \frac{1}{\sqrt{1 + \rho + \rho^2}}$$

and $$\tau' = 1 + \left(\frac{25\rho^2\tau}{(D/t)}\right)\left(\frac{F_y}{0.85f'_c}\right)$$

where $\rho = 0.02(25 - L/D)$

18.8.4 Where bending as well as axial compression is to be resisted, the bending shall be assumed to be resisted by the steel section alone. The steel section shall be proportioned as a beam-column to carry the total bending, plus axial compression equal to the difference between the total axial compression and that portion which can be sustained by the concrete so that:

(a) $\dfrac{M}{\tau M_a} < 1.0$

(b) $\dfrac{P - \tau' P_c}{\tau P_s} + \dfrac{\omega M}{\tau M_s\left(1 - \left(\dfrac{P - \tau' P_c}{P_e}\right)\right)} < 1.0$

when $P > \tau' P_c$

19. General Requirements for Built-Up Members

19.1 General Requirements for Compression Members

19.1.1 All components of built-up compression members and the transverse spacing of their lines of connecting bolts or welds shall meet the requirements of Clause 10 and 11.

19.1.2 All component parts in contact with one another at the ends of built-up compression members shall be connected by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1.5 times the width of the member, or by continuous welds having a length not less than the width of the member.

19.1.3 Unless closer spacing is required for transfer of load, or for sealing inaccessible surfaces, the longitudinal spacing, in line, between intermediate bolts or clear longitudinal spacing between intermittent welds in built-up compression members shall not exceed the following, as applicable:

(a) For compression members composed of two or more rolled shapes in contact or separated from one another by intermittent fillers, the slenderness ratio of any shape between points of interconnection shall not exceed the slenderness ratio of the built-up member. The least radius of gyration of each component part shall be used in computing the slenderness ratio of that part between points of interconnection with other component parts;

(b) $330t/\sqrt{F_y}$ but not more than 300 mm for the outside component of the section consisting of a plate when the bolts on all gauge lines or intermittent welds along the component edges are not staggered, where t = thickness of outside plate;

(c) $525t/\sqrt{F_y}$ but not more than 450 mm for the outside component of the section consisting of a plate when the bolts or intermittent welds are staggered on adjacent lines, where t = thickness of outside plate.

19.1.4 The spacing requirements of Clauses 19.1.3, 19.2.3 and 19.2.4 will not always provide a continuous tight fit between components in contact. When the environment is such that corrosion could be a serious problem, the spacing of bolts or welds may need to be less than the specified maximum.

19.1.5 Open sides of compression members built up from plates or shapes shall be connected to each other by lacing, batten plates, or perforated cover plates.

19.1.6 Lacing shall provide a complete triangulated shear system and may consist of bars, rods or shapes. The spacing of connections of lacing to a main component shall be such that the slenderness ratio of a main component between these points of connection does not exceed the governing slenderness ratio of the member as a whole. Lacing shall be proportioned to resist a shear, normal to the longitudinal axis of the member, of not less than 2.5 per cent of the total axial load on the member plus the shear from transverse loads, if any.

19.1.7 The slenderness ratio of lacing members shall not exceed 140. The effective length for single lacing shall be the distance between connections to the main components; for double lacing connected at the intersections, the effective length shall be 70 per cent of that distance.

19.1.8 Lacing members shall preferably be inclined to the longitudinal axis of the built-up member at an angle of not less than 45°.

19.1.9 Lacing systems shall have diaphragms in the plane of the lacing and as near the ends as practicable and at intermediate points where lacing is interrupted. Such diaphragms may be plates (tie plates) or shapes.

19.1.10 End tie plates used as diaphragms shall have a length not less than the distance between the lines of bolts or welds connecting them to the main components of the member. Intermediate tie plates shall have a length not less than one-half that prescribed for end tie plates. The thickness of tie plates shall be at least 1/60 of the width between lines of bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or clear longitudinal spacing between welds shall not exceed 150 mm. At least three bolts shall connect the tie plate to each main component, or, alternatively, a total length of weld not less than one-third the length of tie plate shall be used.

19.1.11 Shapes used as diaphragms shall be proportioned and connected to transmit from one main component to the other a longitudinal shear equal to 5 per cent of the axial compression in the member.

19.1.12 Perforated cover plates may be used in lieu of lacing and tie plates on open sides of built-up compression members. The net width of such plates at access holes shall be assumed available to resist axial load provided that:

(a) The width-thickness ratio conforms to Clause 11;

(b) The length of the access hole does not exceed twice its width;

(c) The clear distance between access holes in the direction of load is not less than the transverse distance between lines of bolts or welds connecting the perforated plate to the main components of the built-up member;

(d) The periphery of the access hole at all points has a minimum radius of 40 mm.

19.1.13 Battens consisting of plates or shapes may be used on open sides of built-up compression members which do not carry primary bending in addition to axial load. Battens shall be provided at the ends of the member, at locations where the member is laterally supported along its length and elsewhere as determined by the following spacing requirements:

(a) When the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens is equal to or less than 80 per cent of the slenderness ratio with respect to the axis parallel to the battens, the spacing between battens shall be such that the slenderness ratio of a main component between ends of adjacent batten plates shall not exceed 50, nor shall it exceed 70 per cent of the slenderness ratio of the built-up member with respect to the axis parallel to the battens;

(b) When the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens is more than 80 per cent of the slenderness ratio with respect to the axis parallel to the battens, the spacing between battens shall be such that the slenderness ratio of a main component between ends of adjacent batten plates shall not exceed 40, nor shall it exceed 60 per cent of the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens.

19.1.14 Battens shall have a length not less than the distance between lines of bolts or welds connecting them to the main components of the member and a thickness not less than 1/60 of this distance if the batten consists of a flat plate. Battens and their connections shall be proportioned to resist simultaneously a longitudinal shear force, V, and a moment, M.

where

$$V = \frac{0.025Pd}{na} \text{ (Newtons)}$$

$$M = \frac{0.025Pd}{2n} \text{ (N·mm)}$$

d = longitudinal distance centre-to-centre of battens (mm)
a = distance between lines of bolts or welds connecting the batten to each main component (mm)
n = number of parallel planes of battens

19.2 General Requirements for Tension Members

19.2.1 Tension members composed of two or more shapes, plates or bars separated from one another by intermittent fillers shall have the components interconnected at fillers spaced so that the slenderness ratio of any component between points of interconnection shall not exceed 300.

19.2.2 Tension members composed of two plate components in contact or a shape and a plate component in contact shall have the components interconnected so that the spacing between connecting bolts or clear spacing between welds does not exceed 36 times the thickness of the thinner plate nor 450 mm (see Clause 19.1.3).

19.2.3 Tension members composed of two or more shapes in contact shall have the components interconnected so that the spacing between connecting bolts or the clear spacing between welds does not exceed 600 mm, except where it can be determined that a greater spacing would not affect the satisfactory performance of the member (see Clause 19.1.3).

19.2.4 Tension members composed of two separated main components may have either perforated cover plates or tie plates on the open sides of the built-up member. Tie plates, including end tie plates, shall have a length not less than two-thirds of the transverse distance between bolts or welds connecting them to the main components of the member, and shall be spaced so that the slenderness ratio of any component between the tie plates does not exceed 300. The thickness of tie plates shall be at least 1/60 of the transverse distance between the bolts or welds connecting them to the main components and the longitudinal spacing of the bolts or welds shall not exceed 150 mm. Perforated cover plates shall comply with the requirements of Clause 19.1.12(b), (c), and (d).

19.3 General Requirements for Open Box-Type Beams and Grillages

Where two or more rolled beams or channels are used side-by-side to form a flexural member, they shall be connected together at intervals of not more than

1500 mm. Through bolts and separators may be used, provided that in beams having a depth of 300 mm or more, no fewer than two bolts shall be used at each separator location. When concentrated loads are carried from one beam to the other, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be bolted or welded between the beams. The design of members shall provide for torsion resulting from any unequal distribution of loads. Where beams are exposed, they shall be sealed against corrosion of interior surfaces, or spaced sufficiently far apart to permit cleaning and painting.

20. Stability of Structures and Individual Members

20.1 General

20.1.1 In the design of a steel structure care shall be taken to ensure that the structural system is adequate to resist the forces caused by the lateral and vertical loads and to ensure that a complete structural system is provided to transfer the loads to the foundations, particularly when there is a dependence on walls, floors, or roofs acting as shear resisting elements or diaphragms. (See also Clause 8.6.)

Note: The structure should also be checked to ensure that adequate resistance to torsional deformations has been provided.

20.1.2 Design drawings shall indicate all load resisting elements essential to the integrity of the completed structure and shall show details necessary to ensure the effectiveness of the load resisting system. Design drawings shall also indicate the requirements for roofs and floors used as diaphragms.

20.1.3 Erection diagrams shall indicate all load resisting elements essential to the integrity of the completed structure. Permanent and temporary load resisting elements essential to the integrity of the partially completed structure shall be clearly specified on the erection diagrams.

20.1.4 Where the portion of the structure under consideration does not provide adequate resistance to lateral forces, provision shall be made for transferring the forces to adjacent lateral load resisting elements.

20.2 Stability of Columns

Beam-to-column connections shall have adequate strength to transfer the lateral forces produced by possible out-of-plumbness as specified in Clause 29.9.1. These forces shall be computed for the loading cases of Clause 7.2.3 using the appropriate load combination factors. The lateral forces produced by out-of-plumbness shall be computed using 1.7 times the axial force in the column produced by the specified loads.

20.3 Stability of Beams, Girders and Trusses

20.3.1 Bracing members assumed to provide lateral support to the compression flange of beams and girders, or to the compression chord of trusses, and the connections of such bracing members, shall be proportioned to resist a force equal to 1 per cent of the force in the compression flange or chord at the point of support.

20.3.2 When bracing of the compression flange or chord is effected by a slab or deck, the slab or deck and the means by which the computed bracing forces are transmitted between the flange or chord and the slab or deck shall be adequate to resist a force in the plane of the slab or deck. This force shall be considered to be uniformly distributed along the length of the compression flange or chord, and shall be taken as at least 5 per cent of the maximum force in the flange or chord, unless a lesser amount can be justified by analysis.

20.3.3 Consideration shall be given to the probable accumulation of forces when a bracing member must transfer forces from one braced member to another.

20.3.4 Members restraining beams and girders designed to resist loads causing torsion shall be proportioned according to the requirements of Clause 16.11. Special consideration shall be given to the connections of asymmetric section such as channels, angles and zees.

21. Connections

21.1 Alignment of Members
Axially loaded members meeting at a joint shall have their gravity axes intersect at a common point if practicable; otherwise the results of bending due to the joint eccentricity shall be provided for.

21.2 Unrestrained Members
Except as otherwise indicated on the design drawings, all connections of beams, girders, and trusses shall be designed and detailed as flexible

and ordinarily may be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action at the specified load levels in the connection is permitted.

21.3 **Restrained Members**

When beams, girders, or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load. When beams are rigidly framed to the flange of an H-type column, stiffeners shall be provided on the column web as follows:

(a) Opposite the compression flange of beam when

$$w_c\ (t_b + 5k)\ F_{yc} < F_{yb}\ A_f$$

except that for members with Class 3 or 4 webs, the specified load taken by the weld in bearing shall not be considered to exceed

$$\frac{380\ 000}{(h_c/w_c)^2}\ w_c\ (t_b + 5k)$$

(b) Opposite the tension flange of beam when

$$7t_c^{\,2} F_{yc} < F_{yb}\ A_f$$

where

w_c = thickness of column web
t_b = thickness of beam flange
k = distance from outer face of column flange to web toe of fillet, or to web toe of flange-to-web weld in a welded column
F_{yb} = specified yield point of beam flange
F_{yc} = specified yield point of column
h_c = clear depth of column web
t_c = thickness of column flange

The area of a stiffener or pair of stiffeners (A_{st}) opposite either beam flange shall be such that:

$$A_{st} > [A_f \frac{(F_{yb})}{(F_{yc})} - w_c(t_b + 5k)]\ \frac{F_{yc}}{F_{ys}}$$

except that for members with class 3 or 4 webs

$$A_{st} > \left[A_f \frac{(F_{yb})}{(F_{yc})} - \frac{640,000}{(h_c/w_c)^2} w_c (t_b + 5k)\right] \frac{F_{yc}}{F_{ys}}$$

where F_{ys} = specified yield point of a stiffener

Stiffeners shall also be provided on the web of columns, beams or girders if V_r computed from Clause 14.4.2 is exceeded, in which case the stiffener or stiffeners must transfer a shear force equal to:

$$V_{st} = V_f - 0.33wdF_y$$

In all cases the stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one face of the column only, the stiffeners need not be longer than one-half the depth of the column.

21.4 Connections of Tension or Compression Members

The connections at ends of tension members or compression members not finished to bear shall develop the force due to the loads. However the connection shall be designed for not less than 50 per cent of the capacity of the member based on the condition (tension or compression) that governs the selection of the member.

21.5 Bearing Joints in Compression Members

21.5.1 Where columns bear on bearing plates, or are finished to bear at splices, there shall be sufficient fasteners or welds to hold all parts securely in place.

21.5.2 Where other compression members are finished to bear, the splice material and connecting fasteners or welds shall be arranged to hold all parts in place and shall be proportioned for 50 per cent of the computed load.

21.6 Lamellar Tearing

Corner or "T" joint details of rolled structural members, or plates involving transfer of tensile forces in the through-thickness direction resulting from shrinkage due to welding executed under conditions of restraint, shall be avoided where possible. If this type of connection cannot be avoided, measures shall be taken to minimize the possibility of lamellar tearing.

21.7 Placement of Fasteners and Welds

Except in members subjected to repeated loads (as defined in Clause 15), disposition of fillet welds to balance the forces about the neutral axis or axes for end-connections of single angle, double angle, or similar types of axially loaded members is not required. Eccentricity between the gravity axes of such members and the gauge lines of bolted end-connections also may be neglected. In axially loaded members subjected to repeated loads, the fasteners or welds in end connections shall have their centre of gravity on the gravity axis of the member unless provision is made for the effect of the resulting eccentricity.

21.8 Fillers

21.8.1 When load-carrying fasteners pass through fillers with a total thickness greater than 6 mm in bearing-type shear connections, the fillers shall be extended beyond the splice material and the filler extension shall be secured by sufficient fasteners to distribute the total force in the member uniformly over the combined section of the member and the filler, or alternatively an equivalent number of fasteners shall be included in the connection.

21.8.2 In welded construction, any filler with a total thickness greater than 6 mm shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler, as an eccentric load. Welds connecting the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler 6 mm or less in thickness shall have its edges made flush with the edges of the splice plate and the required weld size shall be equal to the thickness of the filler plate plus the size necessary to transmit the splice plate load.

21.9 Welds in Combination

If two or more of the general types of weld (groove, fillet, plug, or slot) are combined in a single connection, the effective capacity of each shall be separately computed with reference to the axis of the group in order to determine the capacity of the combination.

21.10 Fasteners and Welds in Combination

21.10.1 When approved by the designer, high-strength bolts in slip-resistant connections may be considered as sharing the specified load with welds in the same shear plane in new work. In this case, the resistance of the connection, as given by the larger of the individual capacities (high-strength bolts or welds), must also be equal to or greater than the effect of the loads.

21.10.2 In making alterations to structures, existing rivets and high-strength bolts may be utilized to carry forces resulting from existing dead loads, and welding may be proportioned to carry all additional loads.

21.11 High-Strength Bolts (in Slip-Resistant Joints) and Rivets, in Combination

In making alterations, rivets and high-strength bolts in slip-resistant joints may be considered as sharing forces due to dead and live loads.

21.12 Connections Requiring High-Strength Bolts or Welds

21.12.1 High-strength bolts or welds shall be used for the following connections:

(a) Connections of beams, girders, and trusses on which the bracing of the structure is dependent, and column splices, in structures over 30 000 mm in height;

(b) Roof truss splices, connections of trusses to columns, column splices, column bracing, knee braces, and crane supports in all structures carrying cranes over 50 kN capacity;

(c) Connections for supports of running machinery, or of other live loads which produce impact or cyclic load;

(d) Any other connections so stipulated on the design drawings.

21.12.2 In all cases except those listed in Clause 21.12.1 connections may be made with A307 bolts.

21.12.3 For the purposes of Clause 21.12, the height of a tier structure is the distance from curb level to the top of the roof beams in flat roofs or curb level to top of roof beams at mean gable height in the case of sloping roofs. Penthouses may be excluded in determining the height of a structure.

21.13 Special Fasteners

Fasteners of special types may be used when approved by the designer.

22. Bolting Details

22.1 High-Strength Bolts

A325M, A490M, A325 and A490 high-strength bolts and their usage shall conform to Clause 23.

22.2 A307 Bolts

Nuts on A307 bolts shall be tightened to an amount corresponding to the full effort of a man using a spud wrench. When so specified, nuts shall be prevented from working loose by the use of lock washers, lock nuts, jam nuts, thread burring, welding, or other methods approved by the designer.

22.3 Effective Bearing Area

The effective bearing area of bolts shall be the nominal diameter multiplied by the length in bearing. For countersunk bolts half the depth of the countersink shall be deducted from the bearing length.

22.4 Long Grips

A307 bolts which carry calculated loads and with a grip exceeding five diameters shall have their number increased by 0.6 per cent for each additional 1 mm in the grip.

22.5 Minimum Pitch

The minimum distance between centres of bolt holes preferably shall be not less than 3 bolt diameters and in no case less than 2 2/3 diameters.

22.6 Minimum Edge Distance

The minimum distance from the centre of a bolt hole to any edge shall be that given in Table 14.

22.7 Maximum Edge Distance

The maximum distance from the centre of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the outside connected part with a maximum of 150 mm.

22.8 Minimum End Distance

In the connection of tension members having more than two bolts in a line parallel to the direction of load, the minimum end distance (from centre of end fastener to nearest end of connected part) shall be governed by the edge distance values given in Table 14. In members having either one or two bolts in the line of load, the end distance shall be not less than 1.5 bolt diameters.

Table 14
Minimum Edge Distance for Bolt Holes

Bolt Diameter		At Sheared Edge	At Rolled, Sawn or Gas Cut Edge†
Inches*	**Millimetres**	**Millimetres**	**Millimetres**
5/8	-	28	22
-	16	28	22
3/4	-	32	25
-	20	34	26
7/8	-	38‡	28
-	22	38	28
-	24	42	30
1	-	44‡	32
-	27	48	34
1 1/8	-	51	38
-	30	52	38
1 1/4	-	57	41
-	36	64	46
Over 1 1/4	Over 36	1 3/4 x Diameter	1 1/4 x Diameter

* ASTM Standards A325 and A490 are written in Imperial Units. Accordingly, bolt diameters are shown in the Imperial System for these bolts only.
† Gas cut edges shall be smooth and free from notches. Edge distance in this column may be decreased 3 mm when hole is at a point where computed stress is not more than 0.2 of the yield stress.
‡ At ends of beam framing angles this distance may be 32 mm.

22.9 Slotted Holes

Maximum and minimum edge distance for bolts in slotted or oversize holes (as permitted in Clause 23.3.2) shall conform to the requirements given in Clauses 22.6, 22.7, and 22.8 assuming that the fastener can be placed at any extremity of the slot or hole.

23. Structural Joints Using ASTM A325M, A490M, A325 or A490 Bolts

23.1 General

23.1.1 Clause 23 deals with the design, assembly, and inspection of structural joints using ASTM A325M, A490M, A325 or A490 bolts, or equivalent fasteners, tightened to a specific minimum tension. A325M, A490M, A325 and A490 bolts are used in holes slightly larger than the nominal bolt size.

23.1.2 Joints required to resist shear between connected parts shall be designated on design drawings and shop details as either bearing-type or slip-resistant.

23.1.3 Slip-resistant shear joints, in which specified load is assumed to be transferred by the slip resistance of the clamped faying surfaces, shall be required where slippage into bearing cannot be tolerated. Such situations may arise in structures subject to fatigue, frequent load reversal, or sensitive to deflection.

23.1.4 In bearing-type shear joints due recognition of the presence or absence of threads in the shear planes of the joint shall be made. Where an outside part adjacent to a nut is less than 10 mm thick, threads shall be considered to be present unless special precautions are taken.

23.1.5 **Applied Tension**
Bolts required to support load by direct tension shall be proportioned so that the tensile load on the bolt area, independent of initial tightening stress, shall not exceed the tensile stress as given in Clause 13.10.2. The applied load shall be taken as the sum of the external load plus any tension caused by prying action due to deformation of the connected parts. If the connection is subject to repeated loading, prying forces must be avoided.

23.1.6 Joints subject to repeated loads shall be proportioned in accordance with Clause 15.

23.2 Bolts, Nuts and Washers*

23.2.1 Except as provided in Clause 23.2.4, bolts, nuts, and washers shall conform to ASTM Standards: A325M, High-Strength Bolts for Structural Steel Joints (Metric); A490M, High-Strength Steel Bolts, Classes 10.9 and 10.9.3 for Structural Steel Joints (Metric); A325, High-Strength Bolts for Structural Steel Joints; A490, Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength.

***Note:** Before specifying metric bolts, the designer should check on their current availability in the quantities required.

23.2.2 The length of bolts shall be such that the point of the bolt will be flush with, or outside the face of, the nut when completely installed.

23.2.3 If required, A325M and A325 bolts, nuts, and washers may be galvanized in accordance with the requirements of ASTM Standards A325M and A325. When installed on a galvanized bolt in a solid steel connection and with three to five threads in the grip, they shall be capable of producing a tensile-type fracture of the bolt and of rotating one full turn from snug before failure.

23.2.4 Other fasteners which meet the chemical composition requirements of ASTM Standards A325M, A490M, A325, or A490 and which meet the mechanical requirements of the same Standard in full-size tests and which have body diameter and bearing areas under the head and nut, or their equivalent, not less than those provided by a bolt and nut of the same nominal dimensions prescribed by Clause 23.2.1 may be used. Such alternative fasteners may differ in other dimensions from the prescribed bolt and nut dimensions. When such fasteners are proposed as an alternative to A325M, A490M, A325 or A490 standard bolts their use shall be subject to the approval of the designer.

23.2.5 If necessary, washers may be clipped on one side to a point not closer than 7/8 of the bolt diameter from the centre of the washer hole.

23.2.6 Design drawings shall indicate the type or types of bolt which may be used. Shop details and erection diagrams shall show the type of bolt to be used.

23.3 Bolted Parts

23.3.1 Bolted parts shall fit together solidly when assembled and shall not be separated by gaskets or any other interposed compressible material.

23.3.2 Holes may be punched, sub-punched or sub-drilled and reamed, or drilled, as permitted by Clause 27.5. The nominal diameter of a hole shall be not more than 2 mm greater than the nominal bolt size, except that, where shown in the design drawings and at other locations approved by the designer, enlarged or slotted holes may be used with high-strength bolts 16 mm in diameter and larger. Joints utilizing enlarged or slotted holes shall be proportioned in accordance with the requirements of Clause 23 and Clauses 13.11 and 13.12 and shall meet the following conditions:

(a) Oversize holes are 4 mm larger than bolts 22 mm and less in diameter, 6 mm larger than bolts 24 mm in diameter, and 8 mm larger than bolts 27 mm and greater in diameter. Oversized holes shall not be used in bearing-type connections but may be used in any or all plies of slip-resistant connections. Hardened washers shall be used under the head and the nut;

(b) Short slotted holes are 2 mm wider than the bolt diameter and have a length which does not exceed the oversize diameter provisions of Item (a) by more than 2 mm. They may be used in any or all plies of slip-resistant or bearing-type connections. The slots may be used without regard to direction of loading in slip-resistant connections but shall be normal to the direction of the load in bearing-type connections. Hardened washers shall be used under the head and the nut;

(c) Long slotted holes are 2 mm wider than the bolt diameter and have a length more than allowed in Item (b) but not more than 2.5 times the bolt diameter and may be used;

(i) In slip-resistant connections without regard to direction of loading. One-third more bolts shall be provided than would be needed to satisfy the requirements of Clause 13.11;

(ii) In bearing-type connections with the long diameter of the slot normal to the direction of loading. No increase in the number of bolts over those necessary in Clause 13.10 is required;

(iii) In only one of the connected parts of either a slip-resistant or bearing-type connection at an individual faying surface;

(iv) Provided that structural plate washers or a continuous bar not less than 8 mm in thickness cover long slots that are in the outer plies of joints. These washers or bars shall have a size sufficient to cover completely the slot after installation.

(d) The above requirement for the nominal diameter of hole may be waived to permit the use of the following bolt diameters and hole combinations in bearing-type of slip-resistant connections:

(i) Either 3/4-inch diameter bolt or an M20 bolt in a 22 mm hole;

(ii) Either 7/8-inch diameter bolt or an M22 bolt in a 24 mm hole;

(iii) Either 1-inch diameter bolt or an M24 bolt in a 27 mm hole.

23.3.3 When assembled, all joint surfaces including those adjacent to bolt heads, nuts, and washers shall be free of scale (tight mill scale excepted), burrs, dirt, and foreign material which would prevent solid seating of the parts.

23.3.4 Faying surfaces within slip-resistant joints shall, for the categories given in Table 4, be as follows:

(a) For categories 1, 4 and 7, free of oil, paint, lacquer, or other coatings;

(b) For category 2, vinyl wash treatment applied in accordance with SSPC Paint 27, Basic Zinc Chromate - Vinyl Butyral Wash Primer, to blast-cleaned surfaces;

(c) For category 3 and 5, zinc-rich paints as defined in SSPC PS Guide 12.00. Guide for selecting Zinc-Rich Painting Systems, covering zinc-rich paints with organic and inorganic vehicles applied to blast-cleaned surfaces;

(d) For category 6, sprayed metal coatings applied in accordance with CSA Standard G189, Sprayed Metal Coatings for Atmospheric Corrosion Protection;

(e) For category 9, hot-dip galvanizing, provided that faying surfaces are wire brushed or "brush-off" blast-cleaned after galvanizing and prior to assembly.

Faying surfaces within slip-resistant joints also may be coated by other materials and methods provided that these have been sufficiently tested to establish the performance of full-size similarly coated joints to the satisfaction of the designer.

23.4 Installation

23.4.1 Each bolt shall be tightened to provide, when all bolts in the joint are tight, at least the minimum bolt tension given in Table 15 for the size and type of bolt used.

23.4.2 Threaded bolts shall be tightened in accordance with Clause 23.5 or 23.6. If necessary, tightening may be done by turning the bolt while holding the nut against rotation.

23.4.3 A325M and A325 bolts may be installed without a hardened washer except as required by Clause 23.3.2(a), (b), or (c) for oversize or slotted holes, or by Clause 23.7.4 (when inspection involves the use of an inspection wrench). A490M and A490 bolts shall be installed with a hardened washer. For A325M, A490M, A325 and A490 bolts, the hardened washer when used shall be under the element (nut or bolt head) turned in tightening. When A490M or A490 bolts are used with steel having a specified minimum yield point of less than 280 MPa a hardened washer shall be placed under the bolt head and under the nut.

23.4.4 Bevelled washers shall be used to compensate for lack of parallelism where, in the case of A325M and A325 bolts, an outer face of bolted parts has more than a 5 per cent slope with respect to a plane normal to the bolt axis. In the case of A490M and A490 bolts, bevelled washers shall be used to compensate for any lack of parallelism due to slope of outer faces.

23.5 "Turn-of-Nut" Tightening

23.5.1 After aligning the holes in a joint, sufficient bolts shall be placed and brought to a "snug-tight" condition to ensure that the parts of the joint are brought into full contact with each other. "Snug-tight" is the tightness attained by a few impacts of an impact wrench or the full effort of a man using a spud wrench.

23.5.2 Following the initial snugging operation, bolts shall be placed in any remaining open holes and brought to "snug-tightness". Re-snugging may be necessary in large joints.

23.5.3 When all bolts are "snug-tight" each bolt in the joint then shall be tightened additionally by the applicable amount of nut rotation given in Table 16, with tightening progressing systematically from the most rigid part of the joint to its free edges. During this operation there shall be no rotation of the part not turned by the wrench unless the bolt and nut are match-marked to enable the amount of relative rotation to be determined.

23.6 Tightening by Use of a Direct Tension Indicator

Tightening by this means is permitted, provided that it can be demonstrated by an accurate direct measurement procedure that the bolt has been tightened in accordance with Table 15.

23.7 **Inspection**

23.7.1 The inspector shall determine that the requirements of Clauses 23.2, 23.3, 23.4 and 23.5 are met. Installation of bolts shall be observed to ascertain that a proper tightening procedure is employed. The turned element of all bolts shall be visually examined for evidence that they have been tightened. For bearing-type connections with no bolts subject to tensile or combined shear and tensile loads this inspection is all that is required.

23.7.2 Bolts installed by the "turn-of-nut" method may have tensions exceeding those given in Table 15 but this shall not be cause for rejection.

23.7.3 When bolts are installed in accordance with Clause 23.6 the verification that the bolt has been properly tightened is determined by the direct tension indicator.

23.7.4 For bolts in slip-resistant connections and for bolts subject to tensile or combined shear and tension loads, when there is disagreement concerning the results of inspection of bolt tension in the turn-of-nut method, the following arbitration inspection procedure shall be used unless a different procedure has been specified:

(a) The inspector shall use an inspection wrench which shall be a manual or power torque wrench capable of indicating a selected torque value;

(b) Three bolts of the same grade and diameter as those under inspection, and representative of the lengths and condition of those in the structure, shall be placed individually in a calibration device capable of indicating bolt tension. The surface under the part to be turned in tightening each bolt shall be similar to that under the corresponding part in the structure, i.e., there shall be a washer under the part turned if washers are so used in the structure or, if no washer is used, the material abutting the part turned shall be of the same specification as that in the structure;

(c) When the inspection wrench is a manual wrench, each bolt specified in Item (b) shall be tightened in the calibration device by any convenient means first to an initial tension approximately 15 per cent of the required fastener tension and then to the minimum tension specified for its size in Table 15. Tightening beyond the initial condition must not produce greater nut rotation than that permitted in Table 16. The inspection wrench then shall be applied to the tightened bolt and the torque necessary to turn the nut or head 5° in the tightening direction shall be determined. The average torque measured in the tests of three bolts shall be taken as the job inspection torque to be used in the manner specified in Item (e). The job inspection torque shall be established at least once every working day;

(d) When the inspection wrench is a power wrench it shall first be applied to produce an initial tension approximately 15 per cent of the required fastener tension and then adjusted so that it will tighten each bolt specified in Item (b) to a tension at least 5 but not more than 10 per cent greater than the minimum tension specified for its size in Table 15. This setting of wrench shall be taken as the job inspection torque to be used in the manner specified in Item (e). Tightening beyond the initial condition must not produce greater nut rotation than that permitted in Table 16. The job inspection torque shall be established at least once each working day;

(e) Bolts represented by the sample prescribed in Item (b) which have been tightened in the structure shall be inspected by applying, in the tightening direction, the inspecting wrench and its job inspecting torque to 10 per cent of the bolts but not less than two bolts, selected at random in each connection. If no nut or bolt head is turned by this application of the job inspecting torque, the connection shall be accepted as properly tightened. If any nut or bolt head is turned by the application of the job inspecting torque, this torque shall be applied to all bolts in the connection, and all bolts whose nut or head is turned by the job inspecting torque shall be tightened and reinspected. Alternatively, the fabricator or erector at his option may retighten all of the bolts in the connection and then resubmit the connection for the specified inspection.

Table 15
Bolt Tension

Bolt Diameter		Minimum Bolt Tension* (kN)	
Inches	**Millimeters**	**A325M A325**	**A490M A490**
1/2	-	53	67
5/8	-	85	107
-	16	91	114
3/4	-	125	157
-	20	142	178
7/8	-	174	218
-	22	176	220
-	24	205	257
1	-	227	285
-	27	267	334
1 1/8	-	249	356
-	30	326	408
1 1/4	-	316	454
1 3/8	-	378	538
-	36	475	595
1 1/2	-	458	658

* Equal to 70 per cent of specified minimum tensile strength given in the appropriate ASTM specification, soft converted where appropriate and rounded to nearest kilonewton.

Table 16
Nut Rotation* From Snug-Tight Condition

Disposition of Outer Faces of Bolted Parts	Bolt Length‡	Turn
Both faces normal to bolt axis or one face normal to axis and other face sloped 1:20 (bevel washer not used)†	Up To and Including 4 Diameters	1/3
	Over 4 Diameters and Not Exceeding 8 Diameters or 200 mm	1/2
	Exceeding 8 Diameters or 200 mm	2/3
Both faces sloped 1:20 from normal to bolt axis (bevel washers not used)†	For All Lengths of Bolts	3/4

* Nut rotation is rotation relative to bolt regardless of the element (nut or bolt) being turned. Tolerance on rotation: 30° over or under. For coarse thread heavy hex structural bolts of all sizes and length and heavy hex semi-finished nuts.
† Bevel washers are necessary when A490M or A490 bolts are used.
‡ Bolt length is measured from underside of head to extreme end of point.

24. Welding

24.1 Arc Welding
Arc welding design and practice shall conform to CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

24.2 Resistance Welding
Resistance welding practice and design shall conform to the applicable requirements of CSA Standard W55.2, Resistance Welding Practice. The strength of resistance welded joints shall be taken as established in CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, and the related welding practice shall be in conformance with welding standards approved by the Canadian Welding Bureau under the same CSA Standard.

24.3 Fabricator and Erector Qualification
Fabricators and erectors eligible to assume full responsibility for welded construction covered by this Standard shall be those certified by the Canadian Welding Bureau to the requirements of CSA Standard W47.1, Certification of Companies for Fusion Welding of Steel Structures, for Division 1 or Division 2.1 or CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Building, or both, as applicable. For fusion welded structures, part of the work may be sublet to a Division 3 fabricator or erector; however, full responsibility shall remain with the Division 1 or Division 2.1 fabricator or erector.

25. Column Bases

Note: Where references are made in this clause to strength provisions of CSA Standard CAN3-A23.3 which are written in limit states design format the forces derived from specified loads used in calculation shall be multiplied by the load factors given in that Standard or conservatively multiplied by 1.67.

25.1 Loads
Suitable provision shall be made to transfer column loads and moments to footings and foundations.

25.2 Resistance

25.2.1 Compressive Resistance of Concrete
The compressive resistance of concrete shall be taken as $0.3f'_c$, except that when the supporting surface is wider on all sides than the loaded area,

the compressive resistance of the loaded area may be multiplied by $\sqrt{A_2/A_1}$ but not more than 2 where

A_1 = loaded area

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area

When compression exists over the entire base plate area the bearing pressure on the concrete may be assumed to be uniform over an area equal to the width of the base plate multiplied by a depth equal to d-2e where e is the eccentricity of the column load.

25.2.2 Resistance to Pull-Out

Anchor bolts subject to tensile forces shall be anchored to the foundation unit in such a manner that the required tensile force can be developed. Full anchorage is obtained when the pull-out resistance of the concrete is equal to or larger than the tensile resistance of the bolts. The requirements of CSA Standard CAN3-A23.3 for the transfer of tensile forces from the anchors to the concrete shall be met.

25.2.3 Resistance to Transverse Load

25.2.3.1 Shear resistance may be developed by friction between the base plate and the foundation unit or by bearing of the anchor bolts or shear lugs against the concrete. When shear acts towards a free edge, the requirements of CSA Standard CAN3-A23.3 shall be met.

25.2.3.2 When loads are transferred by friction, the requirements of CSA Standard CAN3-A23.3 shall be met.

25.2.3.3 When shear is transmitted by bearing of the anchor bolts on the concrete the bearing stress on the area equal to the product of the bolt diameter and an assumed depth of 5d shall be taken as:

$F_{br} = 0.50nf'_c$

n = number of anchor bolts in shear

25.2.3.4 When shear is transmitted by bearing of shear lugs on the concrete the requirements of CSA Standard CAN3-A23.3 shall be met.

25.2.4 Moment Resistance

The moment resistance shall be taken as the couple formed by the allowable tensile stress of the anchor bolts determined in accordance with Clause 26.2.1 or 26.2.3 as applicable, and by the concrete compressive resistance determined in accordance with Clause 25.2.1.

25.3 **Finishing**

Column bases shall be finished in accordance with the following requirements:

(a) Steel-to-steel contact bearing surfaces of rolled steel bearing plates shall be finished in such a manner that the requirements of Clauses 27.8, 27.9.7 and 29.7.3 are satisfied. In general, rolled steel bearing plates 50 mm or less in thickness may be used without planing provided a satisfactory contact bearing is obtained; rolled steel bearing plates over 50 mm but not over 100 mm in thickness may be straightened by pressing or by planing on all bearing surfaces, to obtain a satisfactory contact bearing; rolled steel bearing plates, over 100 mm in thickness, shall be planed on all bearing surfaces except as noted in Clause 25.2(c);

(b) Column bases other than rolled steel bearing plates shall be planed on all bearing surfaces except as noted in Clause 25.2(c);

(c) The bottom surfaces of bearing plates and column bases which rest on masonry or concrete foundations and are grouted to ensure full bearing need not be planed.

26. **Anchor Bolts**

26.1 **General**

Anchor bolts shall be designed to resist the effect of uplift forces, bending moments and shears determined in accordance with Clause 7.2. The anchorage of the anchor bolts in the foundation unit shall be such that the required load capacity can be developed. Forces present during construction as well as those present in the finished structure shall be resisted.

26.2 **Allowable Stresses on Bolts**

26.2 **Tension**

The allowable tensile stress on the tensile stress area of an anchor bolt shall be taken as:

$F_t = 0.40F_u$

where

A_n = the tensile stress area of the bolt,

$= \frac{\pi}{4} (D - 0.97p)^2$

p = the pitch of the thread in mm

26.2.2 **Shear**
The allowable shear stress of the anchor bolts shall be taken as:

$F_v = 0.27nF_u$

but not greater than the allowable lateral bearing stress given in Clause 25.2.3.3.

When the bolt threads are intercepted by the shear plane the allowable shear stress shall be taken as 70 percent of V_r.

26.2.3 **Shear and Tension**
An anchor bolt required to develop resistance to both tension and shear shall be proportioned so that:

$$\left(\frac{f_t}{F_t}\right)^2 + \left(\frac{f_v}{F_v}\right)^2 \leq 1$$

where f_v is the portion of the total shear per bolt transmitted by bearing of the anchor bolts on the concrete. See Clause 25.2.3.3.

26.2.4 **Tension and Bending**
An anchor bolt required to develop resistance to both tension and bending shall be proportioned to meet the requirements of Clause 13.8(a). The allowable tensile and flexural stresses F_t and F_b, shall be based on the properties of the cross section at the critical section, F_b shall be taken as $0.4F_y$.

27. Fabrication

27.1 **General**
Unless otherwise specified, workmanship shall be in accordance with prevailing practice and the provisions of Clause 27 shall apply to both shop and field fabrication.

27.2 **Straightness of Material**
Prior to layout of fabrication, rolled material

shall be straight within established rolling mill tolerances. Straightening or flattening shall be done by means that will not injure the material and protective coatings, if present. Sharp kinks and bends shall be cause for rejection.

27.3 Gas Cutting

Gas cutting shall be done by machine where practicable. Gas cut edges shall conform to CSA Standard W59, Welded Steel Construction (Metal-Arc Welding). Re-entrant corners shall be free from notches and shall have the largest practical radii, with a minimum radius of 14 mm.

27.4 Sheared or Gas Cut Edge Finish

27.4.1 Planing or finishing of sheared or gas cut edges of plates or shapes shall not be required unless specifically noted on the drawings or included in a stipulated edge preparation for welding.

27.4.2 The use of sheared edges in the tension area shall be avoided in locations subject to plastic hinge rotation at factored loading. If used, such edges shall be finished smooth by grinding, chipping, or planing.

27.4.3 Burrs shall be removed as required in Clause 23.3.3, and when required for proper fit-up for welding, and when burr creates a hazard during or after construction.

27.4.4 The requirements of Clause 27.4.2 shall be noted on design and shop drawings when applicable.

27.5 Holes for Bolts or Other Mechanical Fasteners

27.5.1 Unless otherwise shown on design drawings or as specified in Clause 23.3.2, holes shall be made 2 mm larger than the nominal diameter of the fastener. Holes may be punched when the thickness of material is not greater than the nominal fastener diameter plus 4 mm. For greater thicknesses holes shall be drilled from the solid or either sub-punched or sub-drilled and reamed. The die for all sub-punched holes or the drill for all sub-drilled holes shall be at least 4 mm smaller than the required diameter of the finished hole. Holes in CSA Standard G40.21-M (Type 700Q) or ASTM Standard A514 steels over 13 mm in thickness shall be drilled.

27.5.2 In locations subject to plastic hinge rotation at factored loading, fastener holes in the tension area shall be sub-punched and reamed or drilled full size.

27.5.3 The requirements of Clause 27.5.2 shall be noted on design and shop drawings where applicable.

27.6 Bolted Construction

27.6.1 Drifting necessary during assembly to align holes shall not distort the metal nor enlarge the holes. Holes in adjacent parts shall match sufficiently well to permit easy entry of bolts. If necessary, holes, except oversize or slotted holes, may be enlarged to admit bolts by a moderate amount of reaming; however, gross mismatch of holes shall be cause for rejection.

27.6.2 Assembly of high-strength bolted joints shall be in accordance with Clause 23.

27.7 Welded Construction

Workmanship and technique in arc-welded fabrication shall conform to those prescribed by CSA Standard W59, Welded Steel Construction (Metal-Arc Welding). The welding practice in resistance welded fabrication shall conform to that required by CSA Standard W55.3, Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings, and approved by the Canadian Welding Bureau.

27.8 Finishing of Bearing Surfaces

Compression joints which depend on contact bearing shall have the bearing surfaces prepared to a common plane by milling, sawing, or other suitable means. Surface roughness shall have a roughness height rating not exceeding 500 (12.5 µm) as defined in CSA Standard B95, Surface Texture (Roughness, Waviness, and Lay), unless otherwise specified.

27.9 Tolerances

27.9.1 Structural members consisting primarily of a single rolled shape shall be straight within the tolerances allowed by CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel, except as specified in Clause 27.9.4.

27.9.2 Built-up bolted structural members shall be straight within the tolerances allowed for rolled wide-flange shapes by CSA Standard G40.20-M, General Requirements for Rolled or Welded Structural Quality Steel, except as specified in Clause 27.9.4.

27.9.3 Dimensional tolerances of welded structural members shall be those prescribed by CSA Standard W59, Welded Steel Construction (Metal-Arc Welding), unless otherwise specified.

27.9.4 Fabricated compression members shall not have a deviation from straightness more than one-thousandth of the axial length between points which are to be laterally supported.

27.9.5 Beams with bow within straightness tolerances shall be fabricated so that after erection the bow due to rolling or fabrication shall be upward.

27.9.6 All completed members shall be free from twists, bends, and open joints. Sharp kinks or bends shall be cause for rejection.

27.9.7 Compression joints which depend upon contact bearing, when assembled during fabrication, shall have at least 75 per cent of the entire contact area in full bearing and the separation of any remaining portion shall not exceed 0.25 mm except adjacent to toes of flanges where a localized separation not exceeding 0.60 mm is permissible.

27.9.8 A variation of 1 mm is permissible in the overall length of members with both ends finished for contact bearing.

27.9.9 Members without ends finished for contact bearing, which are to be framed to other steel parts of the structure, may have a variation from the detailed length not greater than 2 mm for members 10 000 mm or less in length and not greater than 4 mm for members over 10 000 mm in length.

28. Cleaning, Surface Preparation and Priming

28.1 General Requirements

28.1.1 All steelwork, except as exempted by Clauses 28.1.2, 28.1.3, and 28.2 or unless otherwise noted on design drawings or in the job specifications, shall be given one coat of primer or one-coat paint (see Clause 28.5) applied in the shop. The primer or one-coat paint shall be applied thoroughly and evenly to dry clean surfaces by suitable means.

28.1.2 Steelwork to be subsequently concealed by interior building finish need not be given a coat of primer unless otherwise specified (see Clause 6.4.2).

28.1.3 Steelwork to be encased in concrete need not be given a coat of primer. Steelwork designed to act compositely with reinforced concrete and depending on natural bond for interconnection shall not be given a coat of primer.

28.1.4 Steelwork to be shop-primed shall be cleaned of all loose mill scale, loose rust, weld slag, and flux deposit, dirt, and other foreign matter and excessive weld spatter prior to application of the primer. Oil and grease shall be removed by solvent. The fabricator shall be free to use any satisfactory method to clean the steel and prepare the surface for painting unless a specific method of surface preparation is called for.

28.1.5 Primer shall be dry before loading primed steelwork for shipment.

28.1.6 Steelwork not to be shop-primed after fabrication shall be cleaned of oil and grease by solvent cleaners and shall be cleaned of dirt and other foreign matter.

28.2 Requirements for Special Surfaces

28.2.1 Surfaces inaccessible after assembly shall be cleaned, or cleaned and primed, as required by Clause 28.1, prior to assembly. Inside surfaces of enclosed spaces entirely sealed off from any external source of oxygen need not be primed.

28.2.2 In compression members, surfaces finished to bear and assembled during fabrication shall be cleaned before assembly but shall not be primed unless otherwise specified.

28.2.3 Surfaces finished to bear and not assembled during fabrication shall be protected by a corrosion inhibiting coating. The coating shall be of a type that can be readily removed prior to assembly or shall be of a type that makes such removal unnecessary.

28.2.4 Faying surfaces of high-strength bolted slip-resistant joints shall not be primed or otherwise coated except as permitted by Clause 23.

28.2.5 Joints to be field welded and surfaces to which shear connections are to be welded shall be kept free of primer and any other coating which could be detrimental to achieving a sound weldment, except that sheet steel decks may be welded to clean primed steelwork.

28.3 Surface Preparation

Unless otherwise specified, or approved, surface preparation shall be in conformance with one of the following applicable specifications of the Steel Structures Painting Council:

SP 2
Hand Tool Cleaning;

SP 3,
Power Tool Cleaning;

SP 4,
Flame Cleaning of New Steel;

SP 5,
White Metal Blast Cleaning;

SP 6,
Commercial Blast Cleaning;

SP 7,
Brush-Off Blast Cleaning;

SP 10,
Near-White Blast Cleaning.

28.4 **Primer**
Unless otherwise specified, or approved, shop primer shall conform to one of the following standards of the Canadian General Standards Board:

1-GP-14e,
Primer: Red Lead in Oil;

1-GP-40d,
Primer: Structural Steel, Oil Alkyd Type;

1-GP-81e,
Primer, Alkyd, Air Drying and Baking, for Vehicles and Equipment;

1-GP-140c,
Primer: Red Lead, Iron Oxide, Oil Alkyd Type;

1-GP-166a,
Primer: Basic Lead Silico-Chromate, Oil Alkyd Type;

CISC/CPMA 2-75,
A Quick-Drying Primer For Use On Structural Steel.

28.5 **One-Coat Paint**
Unless otherwise specified, or approved, one-coat paint intended to withstand exposure to essentially non-corrosive atmosphere for a period of time not

exceeding 6 months shall conform to CISC/CPMA Standard 1-73a, A Quick-Drying One-Coat Paint For Use On Structural Steel.

29. Erection

29.1 General

The steel framework shall be erected true and plumb within the specified tolerances. Temporary bracing shall be employed wherever necessary to withstand all loads to which the structure may be subject during erection and subsequent construction, including loads due to wind, equipment and operation of same. Temporary bracing shall be left in place undisturbed as long as required for the safety and integrity of the structure (see also Clause 26). The erector shall ensure during erection that an adequate margin of safety exists in the uncompleted structure and members.

29.2 Marking of Members

Erection marks or other suitable means shall be used to identify components. Erection marks which are injurious to the material or to finished surfaces exposed to view shall be avoided.

29.3 Handling

Adequate care shall be taken to avoid damage during handling, especially for long slender members. Injury to protective coatings shall be avoided.

29.4 Temporary Loads

Wherever piles of material, erection equipment, or other loads are carried during erection, suitable provision shall be made to ensure that the loads can be safely sustained during their duration and without permanent deformation or other damage to any member of the steel frame and other building components supported thereby.

29.5 Adequacy of Temporary Connections

As erection progresses, the work shall be securely bolted or welded to take care of all dead load, wind, and erection loads, and to assist in providing structural integrity.

29.6 Alignment

No permanent welding or bolting shall be done until as much of the structure as will be stiffened thereby has been suitably aligned.

29.7 Surface Preparation for Field Welding

The portions of surfaces that are to receive welds shall be thoroughly cleaned of all foreign matter, including paint film.

29.8 **Field Painting**

Unless otherwise specified, the cleaning of steelwork in preparation for field painting, touch-up of shop primer, spot-painting of field fasteners, and general field painting, shall not be considered to be a part of the erection work.

29.9 **Erection Tolerances**

29.9.1 Unless otherwise specified, members of the steel framework shall be considered plumb, level, and aligned if the misalignment does not exceed the following tolerances:

(a) Exterior columns of multi-storey buildings - 1 to 1000; but not more than 25 mm towards nor 50 mm away from the building line in the first 20 storeys plus 2 mm for each additional storey up to a maximum of 50 mm towards or 75 mm away from the building line over the full height of the building;

(b) Columns adjacent to elevator shafts - 1 to 1000; but not more than 25 mm in the first 20 storeys plus 1 mm for each additional storey up to a maximum of 50 mm over the full height of the elevator shaft;

(c) Spandrel beams - 1 to 1000;

(d) All other pieces - 1 to 500.

29.9.2 Shelf angles, sash angles, and lintels specified to be provided with adjustable connections shall be considered within tolerances when each piece is level within a tolerance of 1 to 1000, when adjoining ends of these members are aligned vertically within 2 mm and when the locations of these members vertically and horizontally is within 10 mm of the location established by the dimensions on the drawings.

29.9.3 Column splices and other compression joints which depend upon contact bearing shall, after alignment, have at least 65 per cent of the entire contact area in full bearing and the separations of any remaining portions shall not exceed 0.5 mm except locally at toes of flanges, where a separation of 0.75 mm is permissible; otherwise corrective measures shall be taken.

29.9.4 The fit-up of joints to be field welded shall be within the tolerances shown on the field assembly drawings before welding is begun.

30. **Inspection**

30.1 **General**
Material and workmanship at all times shall be subject to inspection by qualified inspectors representing and responsible to the designer. The inspection shall cover shop work and field erection work to ensure compliance with this Standard.

30.2 **Co-operation**
All inspection insofar as possible shall be made in the fabricator's shop and the fabricator shall co-operate with the inspector, permitting access for inspection to all places where work is being done. The inspector shall co-operate in avoiding undue delay in the fabrication or erection of the steelwork.

30.3 **Rejection**
Material or workmanship not conforming to the provisions of this Standard may be rejected at any time during the progress of work when non-conformance to these provisions is established.

30.4 **Inspection of High-Strength Bolted Joints**
The inspection of high-strength bolted joints shall be performed in accordance with the procedures prescribed in Clause 24.

30.5 **Inspection of Welding**
The inspection of welding shall be in accordance with the applicable clause in CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

30.6 **Identification of Steel by Marking**
In the fabricator's plant, steel used for main components shall at all times be marked to identify its specification (and grade, if applicable). This shall be done by suitable markings or by recognized colour coding except that cut pieces identified by piece mark and contract number need not continue to carry specification identification marking when it has been satisfactorily established that such cut pieces conform to the required material specifications.

31. **Additional Requirements for Plastic Design**

31.1 **General**

In addition to meeting the overall requirements of this Standard the design of structures or portions of structures proportioned on the basis of maximum strength, shall meet the requirements of Clauses 8.5, 13.4.2, 13.5.6 and 13.6 as well as Clause 31.

31.2 Permissible Types of Members and Frames

31.2.1 Simple, continuous or rigidly framed members in any structure containing direct acting bracing or shear walls may be proportioned in accordance with Clause 31, except as noted in Clause 8.5. The bracing system shall meet the requirements of Clauses 20 and 31.10.

31.2.2 One and two-storey rigid frames dependent upon frame stiffness alone to prevent sidesway may be proportioned in accordance with Clause 31, except as noted in Clause 8.5.

31.3 Load Factors

Structures or specified portions of structures proportioned in accordance with Clause 31 shall have sufficient strength, as determined by plastic analysis, to support the specified loads multiplied by a load factor of 1.40 for dead load and 1.70 for live loads. The load combination factors given in Clause 7 are applicable unless otherwise specified.

31.4 Tension Members

Tension members in plastically designed structures shall be proportioned so that

$$P_f \leq A_n F_y$$

where

P_f = factored axial load
A_n = critical net area of member

In addition the ratio A_n/A_g shall be greater than F_y/F_u.

31.5 Compression Members

Compression members in plastically designed structures shall be proportioned so that

$$P_f \leq 1.67 A_g F_a$$

where

P_f = factored axial load
A_g = gross area of member
F_a = appropriate allowable axial stress given in Clause 13.3, based on the largest unbraced slenderness ratio.

31.6 Beam-Columns

31.6.1 Published, recognized methods of beam-column analysis acceptable to the Regulatory Authority may be used to proportion beam-columns in plastically designed structures. In lieu of such methods, the requirements of Clause 31.6.2 shall be followed.

31.6.2 Beam-columns of I-shaped members shall be proportioned so that the following conditions are satisfied.

(a) $$\frac{M_{fx}}{M_{px}} + \frac{M_{fy}}{M_{py}} \leq 1.0$$

(b) $$\frac{P_f}{P_y} + \frac{0.85M_{fx}}{M_{px}} + \frac{0.60M_{fy}}{M_{py}} \leq 1.0$$

(c) $$\frac{P_f}{1.67P_a} + \frac{\omega\ M_{fx}}{M_{px}\left(1 - \frac{P_f}{P_{ex}}\right)} + \frac{\omega\ M_{fy}}{M_{py}\left(1 - \frac{P_f}{P_{ey}}\right)}$$

where

P = product of column area and appropriate allowable axial stress given in Clause 13.3, based on the largest unbraced slenderness ratio

P_f = factored axial load

P_y = product of column area and specified yield point of the steel

M_p = plastic moment of cross section (x-axis or y-axis as noted)

M_f = moment from factored loads at the point under consideration; the maximum moment when used in conjunction with Clause 31.6.2(c) (x-axis or y-axis as noted)

P_e = $\frac{1\ 960\ 000A}{(L/r)^2}$ (x-axis or y-axis as noted)

ω = Coefficient used to determine equivalent uniform bending moment. (See Clause 18 for values of ω)

31.7 Shear

Within the boundaries of two or more members whose webs meet in a common plane, the provision in Clause 13.4.2 is satisfied (no web reinforcement required) when:

$$w \geq \frac{7010M}{AF_y}$$

where

A = gross planar area of the connection web (beam depth x column depth)
M = algebraic sum of clockwise and counter-clockwise moments applied on opposite sides of the web boundary
F_y = specified minimum yield point of steel in the web
w = web thickness

31.8 **Web Crippling**
Web stiffeners are required on a member at a point of load application where a plastic hinge would form. When beams are rigidly framed to the flange of an H-type column, stiffeners shall be provided in accordance with Clause 21.3.

31.9 **Width-Thickness Ratios**
The width-thickness ratios of members that would be subject to compression shall meet the requirements of plastic design sections as given in Clause 11.2.

31.10 **Connections**

31.10.1 All connections which are essential to the continuity assumed as the basis of the design analysis shall be capable of resisting the moments, shears, and axial loads to which they would be subjected by factored loading. Both full loading and partial loading cases shall be examined to determine maximum effects.

31.10.2 Corner connections (haunches), tapered or curved for architectural reasons shall be so proportioned that the full plastic bending strength of the section adjacent to the connection can be developed.

31.10.3 Stiffeners shall be used, as required, to preserve the flange continuity of interrupted members at their junction with other members in a continuous frame. Such stiffeners shall be placed in pairs on opposite sides of the web of the member which extends continuously through the joint.

31.10.4 High-strength bolts and A307 bolts shall be proportioned to resist the forces produced by factored load using allowable stresses equal to 1.70 times those given in Clauses 13 and 24, except as noted in Clause 31.10.6. Welds shall be proportioned to resist the forces produced by factored load, using allowable stresses equal to 1.70 times those given in CSA Standard W59 Specification for Welding of Steel Structures (Metal-Arc Welding).

31.10.5 In general, groove welds are preferable to fillet welds, but their use is not mandatory when the strength of the latter at 1.70 times the allowable stresses given in CSA Standard W59 Specification for Welding of Steel Structures (Metal-Arc Welding) is sufficient to resist the factored load imposed upon the joint.

31.10.6 High-strength bolts shall be proportioned, on the basis of no more than 70 per cent of their specified tensile strength, to resist the tension produced by factored load. High-strength bolts required to resist shear may be used in joints having painted faying surfaces when these joints are of such size that the slip required to produce bearing would not interfere with the formation, under factored load, of the plastic hinges assumed in the design.

as a decimal

(b) creep: increase elastic deflections due to dead loads and long term live loads, as computed in (a), by 15%.

(c) shrinkage of concrete: calculate deflections using a selected free shrinkage strain assuming the beam is bent in single curvature by a constant moment. The shrinkage strain is affected by such factors as age of concrete, ratio of slab volume to surface area, and concrete properties (water/cement ratio, percent fines, entrained air, and cement content), and the restraint provided by steel beam and deck. See Appendix K for methods of computing deflections due to shrinkage strains.

18.3.1.2 The web area of steel sections or web system of steel trusses and joists shall be proportioned to carry the total vertical shear V_f.

18.3.1.3 End connections of steel sections, trusses and joists shall be proportioned to transmit the total end reaction of the composite beam.

18.3.2 **Design Effective Width of Concrete**

18.3.2.1 Slabs or cover slabs extending on both sides of the steel section or joist shall be deemed to have a design effective width, b, equal to the least of:

(a) 0.25 times the composite beam span;

(b) 16 times the overall slab thickness (thickness of cover slab plus steel deck depth), plus the width of the top flange of the steel section or top chord of the steel truss or joist;

(c) The average distance from the centre of the steel section, truss or joist to the centres of adjacent parallel supports.

18.3.2.2 Slabs or cover slabs extending on one side only of the supporting section or joist shall be deemed to have a design effective width, b, not greater than the width of top flange of the steel section, or top chord of the steel joist, plus the least of:

(a) 0.1 times the composite beam span;

(b) 6 times the overall slab thickness or (thickness of cover slab plus steel deck depth);

(c) 0.5 times the clear distance between the steel section or joist and the adjacent parallel support.

18.3.3 Slab Reinforcement

Slabs shall be adequately reinforced to support all specified loads and to control cracking both parallel and transverse to the composite beam span. Reinforcement parallel to the span of the beam in regions of negative bending moment of the composite beam shall be anchored by embedment in concrete which is in compression. The reinforcement of slabs which are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention.

The possibility of longitudinal cracking due to composite action, directly over the steel section or joist, shall be controlled by the provision of additional transverse reinforcement or other effective means unless it is known from experience that cracking due to composite action is unlikely. Such additional reinforcement shall be placed in the lower part of the slab and anchored so as to develop the yield strength of the reinforcement. The area of such reinforcement shall be not less than 0.005 times the concrete area in the longitudinal direction of the beam and shall be uniformly spaced along the composite beam span.

18.3.4 Composite Action With Steel Deck

Cover slabs intended to act compositely with steel deck shall have reinforcement transverse to the span of the composite beam as required. Reinforcement shall be not less than that required by the specified fire resistance design of the assembly.

18.3.5 Interconnection

18.3.5.1 Except as permitted by Clauses 18.3.5.2 and 18.3.5.4 interconnection between steel sections, trusses or joists and slabs or cellular steel deck with cover slabs shall be attained by the use of shear connectors as prescribed in Clause 18.3.6.

18.3.5.2 Unpainted steel sections, trusses or joists supporting slabs and totally encased in concrete do not require interconnection by means of shear connectors provided that:

(a) A minimum of 50 mm of concrete covers all portions of the steel section or joist, except as noted in Item (c);

(b) The cover in Item (a) is reinforced to prevent spalling; and

(c) The top of the steel section or joist is at least 40 mm below the top and 50 mm above the bottom of the slab.

18.3.5.3 Studs may be welded through a maximum of two steel sheets in contact, each not more than 1.71 mm in overall thickness including coatings (1.52 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise holes for placing studs shall be made through the sheets as necessary. Welded studs shall meet the requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).

18.3.5.4 Other methods of interconnection which have been adequately demonstrated by test and verified by analysis may be used to effect the transfer of forces between the steel section, truss or joist and the slab or steel deck with cover slab. In such cases the design of the composite member shall conform to the design of a similar member employing shear connectors, insofar as practicable.

18.3.5.5 The diameter of a welded stud shall not exceed 2.5 times the thickness of the part to which it is welded, unless test data satisfactory to the designer is provided to establish the capacity of the stud as a shear connector.

18.3.6 **Shear Connectors**

The capacity, q, of a shear connector shall be established by tests acceptable to the designer, except that the following values shall be acceptable without further verification:

(a) **End welded studs, headed or hooked with h/d > 4**

$$q = 0.5A_{sc}\sqrt{f'_c E_c} \leq F_u{}^* A_{sc} \quad \text{(newtons)}$$

$^*F_u$ for commonly available studs is 415 MPa.

This value is limited to designs incorporating a solid concrete slab; or designs incorporating a ribbed slab formed by casting a concrete cover slab on a fluted steel deck in which the rib average width is at least twice the height of the formed concrete rib and the projection of the stud, based on its length prior to welding, is at least two stud diameters above the top surface of the steel deck;

(b) **End welded studs, headed, in selected cases -**

Table 13 gives values of q_r for selected cases of composite beams incorporating steel deck not covered by Item (a). Cover slabs shall consist of normal density concrete (2300 kg/m^3) with $f'_c > 20$ MPa.

(c) **Channel connectors**

$$q = 36.5 \left(t_f + 0.5t_w\right) L_c\sqrt{f'_c} \quad \text{(newtons)}$$

This formula is limited to design incorporating a solid concrete slab of normal density concrete (2300 kg/m^3) with $f'_c > 20$ MPa.

Table 13
Shear Capacities of Studs for Selected Cases

Height of Cellular Steel Deck (mm)	Average Rib Width, Minimum (mm)	Depth of Cover Slab (mm)	Stud Size d x h (mm x mm)	No. of Studs per Rib	Shear Capacity* q (newtons)
38/43	50	65	14 x 75	1	24 000
38/43	50	65	20 x 75	1	50 000
				2	76 000 per pair
38/43	50	90	20 x 100	1	79 000
				2	113 000 per pair

* Shear capacities given in Table 13 are derived from test and reflect the influence of rib geometry and stiffness on the useful capacity of the studs.

18.3.7 **Ties**
Mechanical ties shall be provided between the steel section, truss or joist and the slab or steel deck to prevent separation. Shear connectors may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1000 mm and the average spacing in a span should not exceed 600 mm nor be greater than that required to achieve any specified fire resistance rating of the composite assembly.

18.4 **Design of Composite Beams With Shear Connectors**

18.4.1 The composite beam shall consist of steel section, truss or joist, shear connectors, ties, and slab or steel deck with cover slab.

18.4.2 The properties of the composite section shall be computed according to elastic theory neglecting any concrete area in tension within the maximum effective area equal to effective width times effective thickness. If a steel truss or joist is used the area of its top chord shall be neglected in determining the properties of composite section. The effective area of concrete in compression shall be considered as an equivalent area of steel by dividing it by the appropriate modular ratio.

18.4.3 The composite section shall be proportioned to resist the total specified load without exceeding the allowable stresses for steel and concrete respectively.

18.4.4 If fewer than the number of shear connectors required for full composite action are to be provided (see Clause 18.4.6) the effective area of concrete in compression to be assumed in design shall be reduced from that given in Clause 18.2 and 18.3 in proportion to the ratio n'/n.

where

n' = number of shear connectors provided
n =number of shear connectors required for full composite action according to Clause 18.4.6.

18.4.5 No composite action shall be assumed in computing flexural strength when $n' < 0.5n$.

No composite action shall be assumed in computing deflections when $n' < 0.25n$.

18.4.6 For full shear connection, the total horizontal shear, V_h, at the junction of the steel section or joist and the concrete slab or steel deck, to be resisted by shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment, shall be the lesser of

$$V_h = A_s F_y$$

$$V_h = 0.85btf'_c$$

18.4.7 Composite beams employing steel sections and concrete slabs may be designed as continuous members. The moment capacity of the composite section with the concrete slab in the tension area of the composite section shall be the moment resistance of the steel section alone except that when sufficient shear connectors are placed in the negative moment region, suitably anchored concrete slab reinforcement parallel to the steel sections and within the design effective width of the concrete slab may be included in computing the properties of the composite section. The total horizontal shear, V_h, to be resisted by shear connectors between the point of maximum negative bending moment and the adjacent point of zero moment shall be taken as:

$$V_h = A_r F_{yr}$$

18.4.8 The number of shear connectors to be located each side of the point of maximum bending moment (positive or negative, as applicable) and distributed between that point and the adjacent point of zero moment shall be not less than:

$$n = \frac{V_h}{q}$$

Shear connectors may be spaced uniformly except that in a region of positive bending, the number of shear connectors required between any concentrated load applied in that region and the nearest point of zero moment shall be not less than n"

$$n'' = n \left(\frac{M - M_s}{M_{max} - M_s}\right)$$

where

M = positive bending moment at concentrated load point
M_s = moment capacity of the steel section alone
M_{max} = maximum positive bending moment

18.5 Design of Composite Beams Without Shear Connectors

18.5.1 Unpainted steel sections or joists supporting concrete slabs and encased in concrete in accordance with Clause 18.3.5.2 may be proportioned on the basis that the composite section supports the total load.

18.5.2 In computing the moment of inertia of the composite section any concrete area in tension shall be neglected.

18.5.3 The maximum bending stress in the steel section or joist shall not exceed the allowable bending stresses given in Clause 13.5 for the class of section used and the maximum compressive bending stress in the concrete shall not exceed $0.45f'_c$.

18.6 Unshored Beams

For composite beams unshored during construction, the stresses in the tension flange of the steel section or joist due to the loads applied before the concrete strength reaches $0.75f'_c$ plus the stresses at the same location, due to the remaining specified loads considered to act on the composite section shall not exceed $0.90F_y$.

18.7 **Beams During Construction**

The steel section or joist alone shall be proportioned to support all loads applied prior to hardening of the concrete without exceeding its calculated capacity under the conditions of lateral support and shoring, as applicable, to be furnished during construction.

18.8 **Design of Composite Columns (Concrete-Filled Hollow Structural Sections)**

18.8.1 Hollow structural sections designated as Class 1, 2 or 3 sections which are completely filled with concrete may be assumed to carry compressive load as composite columns. Class 4 hollow structural sections completely filled with concrete may also be designed as composite columns providing the width-thickness ratios of the walls of rectangular sections do not exceed $1350/\sqrt{F_y}$, and the outside diameter to thickness ratio of circular sections do not exceed $28\,000/F_y$.

18.8.2 The proportion of the axial load assumed to be carried by the concrete shall be applied by direct bearing on the concrete, or, alternatively, other methods of load application to the concrete may be employed if their adequacy has been demonstrated by test.

18.8.3 The allowable axial compressive load of a composite column shall be taken as:

$$P_a = \tau P_s + \tau' P_c$$

where

P_s = allowable axial load on steel section alone

$= A_s F_a$ with F_a defined in Clause 13.3

P_c = allowable axial load on concrete area A_c

$$P_c = 0.35\, f'_c A_c \lambda^{-2}\left[\sqrt{1 + 0.25\lambda_c^{-4}} - 0.5\lambda_c^{-2}\right]$$

in which $\lambda_c = \dfrac{KL}{r_c}\sqrt{\dfrac{f'_c}{\pi^2 E_c}}$

r_c = radius of gyration of the concrete area, A_c,

E_c = initial elastic modulus for concrete in MPa, considering the effects of long term loading. For normal weight concrete, with f'_c expressed in MPa, this may be taken as:

$$(1 + S/T)\ 2500 \sqrt{f'_c}$$

where S is the short term load and T is the total load on the column.

For all rectangular hollow structural sections, and for circular hollow structural sections with height to diameter ratio of 25 or greater, $\tau = \tau' = 1.0$

Otherwise $\tau = \dfrac{1}{\sqrt{1 + \rho + \rho^2}}$

and $\tau' = 1 + \left(\dfrac{25\rho^2\tau}{(D/t)}\right)\left(\dfrac{F_y}{0.85f'_c}\right)$

where $\rho = 0.02(25 - L/D)$

18.8.4 Where bending as well as axial compression is to be resisted, the bending shall be assumed to be resisted by the steel section alone. The steel section shall be proportioned as a beam-column to carry the total bending, plus axial compression equal to the difference between the total axial compression and that portion which can be sustained by the concrete so that:

(a) $\dfrac{M}{\tau M_a} \leqslant 1.0$

(b) $\dfrac{P - \tau' P_c}{\tau P_s} + \dfrac{\omega M}{\tau M_s\left(1 - \left(\dfrac{P - \tau' P_c}{P_e}\right)\right)} \leqslant 1.0$

when $P > \tau' P_c$

19. General Requirements for Built-Up Members

19.1 General Requirements for Compression Members

19.1.1 All components of built-up compression members and the transverse spacing of their lines of connecting bolts or welds shall meet the requirements of Clause 10 and 11.

19.1.2 All component parts in contact with one another at the ends of built-up compression members shall be connected by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1.5 times the width of the member, or by continuous welds having a length not less than the width of the member.

19.1.3 Unless closer spacing is required for transfer of load, or for sealing inaccessible surfaces, the longitudinal spacing, in line, between intermediate bolts or clear longitudinal spacing between intermittent welds in built-up compression members shall not exceed the following, as applicable:

(a) For compression members composed of two or more rolled shapes in contact or separated from one another by intermittent fillers, the slenderness ratio of any shape between points of interconnection shall not exceed the slenderness ratio of the built-up member. The least radius of gyration of each component part shall be used in computing the slenderness ratio of that part between points of interconnection with other component parts;

(b) $330t/\sqrt{F_y}$ but not more than 300 mm for the outside component of the section consisting of a plate when the bolts on all gauge lines or intermittent welds along the component edges are not staggered, where t = thickness of outside plate;

(c) $525t/\sqrt{F_y}$ but not more than 450 mm for the outside component of the section consisting of a plate when the bolts or intermittent welds are staggered on adjacent lines, where t = thickness of outside plate.

19.1.4 The spacing requirements of Clauses 19.1.3, 19.2.3 and 19.2.4 will not always provide a continuous tight fit between components in contact. When the environment is such that corrosion could be a serious problem, the spacing of bolts or welds may need to be less than the specified maximum.

19.1.5 Open sides of compression members built up from plates or shapes shall be connected to each other by lacing, batten plates, or perforated cover plates.

19.1.6 Lacing shall provide a complete triangulated shear system and may consist of bars, rods or shapes. The spacing of connections of lacing to a main component shall be such that the slenderness ratio of a main component between these points of connection does not exceed the governing slenderness ratio of the member as a whole. Lacing shall be proportioned to resist a shear, normal to the longitudinal axis of the member, of not less than 2.5 per cent of the total axial load on the member plus the shear from transverse loads, if any.

19.1.7 The slenderness ratio of lacing members shall not exceed 140. The effective length for single lacing shall be the distance between connections to the main components; for double lacing connected at the intersections, the effective length shall be 70 per cent of that distance.

19.1.8 Lacing members shall preferably be inclined to the longitudinal axis of the built-up member at an angle of not less than 45°.

19.1.9 Lacing systems shall have diaphragms in the plane of the lacing and as near the ends as practicable and at intermediate points where lacing is interrupted. Such diaphragms may be plates (tie plates) or shapes.

19.1.10 End tie plates used as diaphragms shall have a length not less than the distance between the lines of bolts or welds connecting them to the main components of the member. Intermediate tie plates shall have a length not less than one-half that prescribed for end tie plates. The thickness of tie plates shall be at least 1/60 of the width between lines of bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or clear longitudinal spacing between welds shall not exceed 150 mm. At least three bolts shall connect the tie plate to each main component, or, alternatively, a total length of weld not less than one-third the length of tie plate shall be used.

19.1.11 Shapes used as diaphragms shall be proportioned and connected to transmit from one main component to the other a longitudinal shear equal to 5 per cent of the axial compression in the member.

19.1.12 Perforated cover plates may be used in lieu of lacing and tie plates on open sides of built-up compression members. The net width of such plates at access holes shall be assumed available to resist axial load provided that:

(a) The width-thickness ratio conforms to Clause 11;

(b) The length of the access hole does not exceed twice its width;

(c) The clear distance between access holes in the direction of load is not less than the transverse distance between lines of bolts or welds connecting the perforated plate to the main components of the built-up member;

(d) The periphery of the access hole at all points has a minimum radius of 40 mm.

19.1.13 Battens consisting of plates or shapes may be used on open sides of built-up compression members which do not carry primary bending in addition to axial load. Battens shall be provided at the ends of the member, at locations where the member is laterally supported along its length and elsewhere as determined by the following spacing requirements:

(a) When the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens is equal to or less than 80 per cent of the slenderness ratio with respect to the axis parallel to the battens, the spacing between battens shall be such that the slenderness ratio of a main component between ends of adjacent batten plates shall not exceed 50, nor shall it exceed 70 per cent of the slenderness ratio of the built-up member with respect to the axis parallel to the battens;

(b) When the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens is more than 80 per cent of the slenderness ratio with respect to the axis parallel to the battens, the spacing between battens shall be such that the slenderness ratio of a main component between ends of adjacent batten plates shall not exceed 40, nor shall it exceed 60 per cent of the slenderness ratio of the built-up member with respect to the axis perpendicular to the battens.

19.1.14 Battens shall have a length not less than the distance between lines of bolts or welds connecting them to the main components of the member and a thickness not less than 1/60 of this distance if the batten consists of a flat plate. Battens and their connections shall be proportioned to resist simultaneously a longitudinal shear force, V, and a moment, M.

where

$$V = \frac{0.025Pd}{na} \text{ (Newtons)}$$

$$M = \frac{0.025Pd}{2n} \text{ (N·mm)}$$

d = longitudinal distance centre-to-centre of battens (mm)
a = distance between lines of bolts or welds connecting the batten to each main component (mm)
n = number of parallel planes of battens

19.2 General Requirements for Tension Members

19.2.1 Tension members composed of two or more shapes, plates or bars separated from one another by intermittent fillers shall have the components interconnected at fillers spaced so that the slenderness ratio of any component between points of interconnection shall not exceed 300.

19.2.2 Tension members composed of two plate components in contact or a shape and a plate component in contact shall have the components interconnected so that the spacing between connecting bolts or clear spacing between welds does not exceed 36 times the thickness of the thinner plate nor 450 mm (see Clause 19.1.3).

19.2.3 Tension members composed of two or more shapes in contact shall have the components interconnected so that the spacing between connecting bolts or the clear spacing between welds does not exceed 600 mm, except where it can be determined that a greater spacing would not affect the satisfactory performance of the member (see Clause 19.1.3).

19.2.4 Tension members composed of two separated main components may have either perforated cover plates or tie plates on the open sides of the built-up member. Tie plates, including end tie plates, shall have a length not less than two-thirds of the transverse distance between bolts or welds connecting them to the main components of the member, and shall be spaced so that the slenderness ratio of any component between the tie plates does not exceed 300. The thickness of tie plates shall be at least 1/60 of the transverse distance between the bolts or welds connecting them to the main components and the longitudinal spacing of the bolts or welds shall not exceed 150 mm. Perforated cover plates shall comply with the requirements of Clause 19.1.12(b), (c), and (d).

19.3 General Requirements for Open Box-Type Beams and Grillages

Where two or more rolled beams or channels are used side-by-side to form a flexural member, they shall be connected together at intervals of not more than

1500 mm. Through bolts and separators may be used, provided that in beams having a depth of 300 mm or more, no fewer than two bolts shall be used at each separator location. When concentrated loads are carried from one beam to the other, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be bolted or welded between the beams. The design of members shall provide for torsion resulting from any unequal distribution of loads. Where beams are exposed, they shall be sealed against corrosion of interior surfaces, or spaced sufficiently far apart to permit cleaning and painting.

20. Stability of Structures and Individual Members

20.1 General

20.1.1 In the design of a steel structure care shall be taken to ensure that the structural system is adequate to resist the forces caused by the lateral and vertical loads and to ensure that a complete structural system is provided to transfer the loads to the foundations, particularly when there is a dependence on walls, floors, or roofs acting as shear resisting elements or diaphragms. (See also Clause 8.6.)

Note: The structure should also be checked to ensure that adequate resistance to torsional deformations has been provided.

20.1.2 Design drawings shall indicate all load resisting elements essential to the integrity of the completed structure and shall show details necessary to ensure the effectiveness of the load resisting system. Design drawings shall also indicate the requirements for roofs and floors used as diaphragms.

20.1.3 Erection diagrams shall indicate all load resisting elements essential to the integrity of the completed structure. Permanent and temporary load resisting elements essential to the integrity of the partially completed structure shall be clearly specified on the erection diagrams.

20.1.4 Where the portion of the structure under consideration does not provide adequate resistance to lateral forces, provision shall be made for transferring the forces to adjacent lateral load resisting elements.

20.2 Stability of Columns

Beam-to-column connections shall have adequate strength to transfer the lateral forces produced by possible out-of-plumbness as specified in Clause 29.9.1. These forces shall be computed for the loading cases of Clause 7.2.3 using the appropriate load combination factors. The lateral forces produced by out-of-plumbness shall be computed using 1.7 times the axial force in the column produced by the specified loads.

20.3 Stability of Beams, Girders and Trusses

20.3.1 Bracing members assumed to provide lateral support to the compression flange of beams and girders, or to the compression chord of trusses, and the connections of such bracing members, shall be proportioned to resist a force equal to 1 per cent of the force in the compression flange or chord at the point of support.

20.3.2 When bracing of the compression flange or chord is effected by a slab or deck, the slab or deck and the means by which the computed bracing forces are transmitted between the flange or chord and the slab or deck shall be adequate to resist a force in the plane of the slab or deck. This force shall be considered to be uniformly distributed along the length of the compression flange or chord, and shall be taken as at least 5 per cent of the maximum force in the flange or chord, unless a lesser amount can be justified by analysis.

20.3.3 Consideration shall be given to the probable accumulation of forces when a bracing member must transfer forces from one braced member to another.

20.3.4 Members restraining beams and girders designed to resist loads causing torsion shall be proportioned according to the requirements of Clause 16.11. Special consideration shall be given to the connections of asymmetric section such as channels, angles and zees.

21. Connections

21.1 Alignment of Members

Axially loaded members meeting at a joint shall have their gravity axes intersect at a common point if practicable; otherwise the results of bending due to the joint eccentricity shall be provided for.

21.2 Unrestrained Members

Except as otherwise indicated on the design drawings, all connections of beams, girders, and trusses shall be designed and detailed as flexible and ordinarily may be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action at the specified load levels in the connection is permitted.

21.3 **Restrained Members**

When beams, girders, or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load. When beams are rigidly framed to the flange of an H-type column, stiffeners shall be provided on the column web as follows:

(a) Opposite the compression flange of beam when

$$w_c\,(t_b + 5k)\,F_{yc} < F_{yb}\,A_f$$

except that for members with Class 3 or 4 webs, the specified load taken by the weld in bearing shall not be considered to exceed

$$\frac{380\ 000}{(h_c/w_c)^2}\,w_c\,(t_b + 5k)$$

(b) Opposite the tension flange of beam when

$$7t_c^{\,2}F_{yc} < F_{yb}\,A_f$$

where

w_c = thickness of column web
t_b = thickness of beam flange
k = distance from outer face of column flange to web toe of fillet, or to web toe of flange-to-web weld in a welded column
F_{yb} = specified yield point of beam flange
F_{yc} = specified yield point of column
h_c = clear depth of column web
t_c = thickness of column flange

The area of a stiffener or pair of stiffeners (A_{st}) opposite either beam flange shall be such that:

$$A_{st} > \left[A_f\,\frac{(F_{yb})}{(F_{yc})} - w_c(t_b + 5k)\right]\frac{F_{yc}}{F_{ys}}$$

except that for members with class 3 or 4 webs

$$A_{st} > \left[A_f \frac{(F_{yb})}{(F_{yc})} - \frac{640,000}{(h_c/w_c)^2} \; w_c \; (t_b + 5k)\right] \frac{F_{yc}}{F_{ys}}$$

where F_{ys} = specified yield point of a stiffener

Stiffeners shall also be provided on the web of columns, beams or girders if V_r computed from Clause 14.4.2 is exceeded, in which case the stiffener or stiffeners must transfer a shear force equal to:

$$V_{st} = V_f - 0.33wdF_y$$

In all cases the stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one face of the column only, the stiffeners need not be longer than one-half the depth of the column.

21.4 Connections of Tension or Compression Members

The connections at ends of tension members or compression members not finished to bear shall develop the force due to the loads. However the connection shall be designed for not less than 50 per cent of the capacity of the member based on the condition (tension or compression) that governs the selection of the member.

21.5 Bearing Joints in Compression Members

21.5.1 Where columns bear on bearing plates, or are finished to bear at splices, there shall be sufficient fasteners or welds to hold all parts securely in place.

21.5.2 Where other compression members are finished to bear, the splice material and connecting fasteners or welds shall be arranged to hold all parts in place and shall be proportioned for 50 per cent of the computed load.

21.6 Lamellar Tearing

Corner or "T" joint details of rolled structural members, or plates involving transfer of tensile forces in the through-thickness direction resulting from shrinkage due to welding executed under conditions of restraint, shall be avoided where possible. If this type of connection cannot be avoided, measures shall be taken to minimize the possibility of lamellar tearing.

21.7 Placement of Fasteners and Welds

Appendix A

Standard Practice for Structural Steel for Buildings

Note: This Appendix is not a mandatory part of this Standard.

A1. Matters concerning standard practice not covered by the Standard but pertinent to the fabrication and erection of structural steel, such as a definition of structural steel items, the computation of weights, etc., should be clearly specified in the plans and specifications issued to the bidders, or in accordance with a specification like the "Code of Standard Practice for Structural Steel" published by the Canadian Institute of Steel Construction.

Appendix B

Effective Lengths of Compression Members in Frames

Note: This Appendix is not a mandatory part of this Standard.

B1. The slenderness ratio of a compression member is defined as the ratio of the effective length to the applicable radius of gyration. The effective length KL may be thought of as the actual unbraced length L multiplied by a factor K such that the product KL is equal to the length of a pin-ended compression member of equal capacity to the actual member. The effective length factor K of a column of finite unbraced length is therefore dependent upon the conditions of restraint afforded to the column at its braced locations and theoretically may vary from 0.5 to infinity. In practical building applications, K would be somewhat greater than 0.5 in the most favourable situation and in all probability would not exceed 5 in the most unfavourable situation.

B2. A variation in K between 0.65 and 2.0 would apply to the majority of cases likely to be encountered in actual structures.

B3. When proportioning columns on the basis of effective lengths the designer is presented with two basic situations which have a pronounced effect upon the strength of axially loaded columns.

(a) For structures in which the sway effects have been included in the analysis to determine the design moments and forces, the effective length factor is determined from the degree of rotational restraint afforded at the ends of the unbraced length and K will be equal to or less than 1.0. In Appendix C this case is identified as the side-sway prevented case;

(b) For structures in which the sway effects have not been included in the analysis to determine the design moments and forces, the effective length factor is determined from the degree of rotational and translational restraint afforded at the ends of the unbraced length and K will be equal to or greater than 1.0. In Appendix C this case is identified as the side-sway permitted case.

B4. Figure B1 illustrates six idealized cases in which joint rotation and translation are either fully realized or non-existent.

B5. For a frame with columns pinned at their bases, Figure B2 shows diagrammatically the difference in effective column length when the sway effect is and is not included in the analysis to determine the design moments and forces. In the former case, the effect of the loads acting at the displacement Δ_b has been included in the analysis and the effective length, K_aL is based on the sway prevented condition ($K_aL < L$) which accounts only for the effect of the displacement Δ_a on column stability. In the latter case, the effective length K_bL is based on the sway permitted condition ($K_bL > L$) in an attempt to include the influence of the displacement Δ_b on column stability.

B6. The use of the sway permitted case is approximate only as the moments and forces due to the sway effects are not taken into account in the design of the girders. In frames that depend on means other than frame action to achieve stability, such as bracing, the bracing shall be designed to resist the combined effects of the superimposed loads plus the loads due to the sway effects.

B7. In Figure B2 the column bases are shown to be pinned and G_L would theoretically be infinity. In practical situations, however, the restraining effect of the normal flat-ended column base detail exerts a beneficial influence on the true effective length of the column, even where the footing is designed only for vertical load. Thus in most cases G_L can be taken as 10 (or less where justified) in the computation of K.

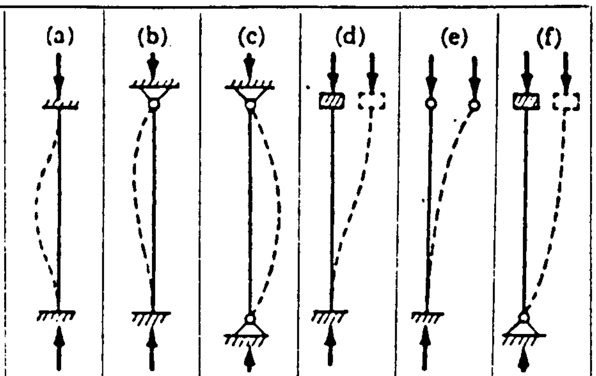
	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.0	2.0
End condition code		Rotation fixed / Translation fixed; Rotation free / Translation fixed; Rotation fixed / Translation free; Rotation free / Translation free				

Figure B1

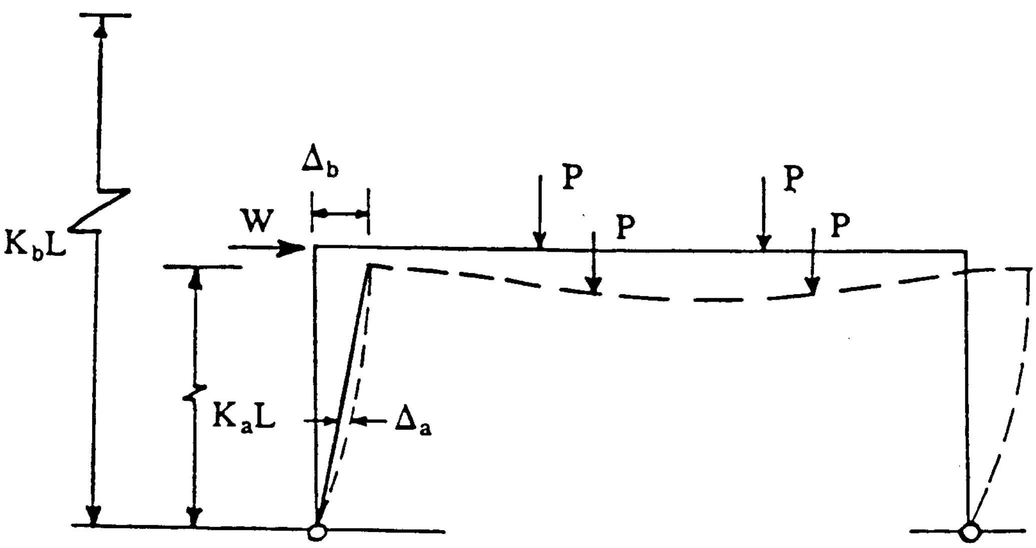


Figure B2

Appendix C

Criteria for Estimating Effective Column Lengths in Continuous Frames

Note: This Appendix is not a mandatory part of this Standard.

C1. Two cases influencing the design of columns in continuous frames are considered:

(a) Sway effects included in the analysis - side-sway prevented;

(b) Sway effects not included in the analysis - side-sway permitted.

C2. Figure C1 is a nomograph applicable to cases in which the equivalent I/L of adjacent girders which are rigidly attached to the columns are known, and is based on the assumption that all columns in the portion of the framework considered reach their individual critical loads simultaneously.

In the usual building frame not all columns would be loaded so as to simultaneously reach their buckling loads, and thus some conservatism is introduced in the interest of simplification.

C3. The equations upon which these nomographs are based are:

(a) Side-sway prevented:

$$\frac{G_U G_L}{4}(\pi/K)^2 + \left(\frac{G_U + G_L}{2}\right)\left(1 - \frac{\pi/K}{\tan \pi/K}\right) + 2\,\frac{\tan \pi/2K}{\pi/K} = 1$$

(b) Side-sway permitted:

$$\frac{G_U G_L(\pi/K)^2 - 36}{6(G_U - G_L)} = \frac{\pi/K}{\tan \pi/K}$$

C4. Subscripts U and L refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\Sigma I_c/L_c}{\Sigma I_g/L_g}$$

in which Σ indicates a summation for all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered, I_c is the moment of inertia and L_c the unsupported length of a column section, and I_g is the moment of inertia and L_g the unsupported length of a girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered.

C5. For column ends supported by, but not rigidly connected to, a footing or foundation, "G" may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, "G" may be taken as 1.0. Smaller values may be used if justified by analysis.

C6. Refinements in girder I_g/L_g may be made when conditions at the far end of any particular girder are known definitely or when a conservative estimate can be made. For the case with no side-sway, multiply girder stiffnesses by the following factors:

1.5 for far end of girder hinged;
2.0 for far end of girder fixed against rotation; (i.e., rigidly attached to a support which is itself relatively rigid).

C7. For the case with side-sway permitted, multiply girder stiffnesses by 0.5 for far end of girder hinged.

C8. Having determined G_U and G_L for a column section, the effective length factor K is determined by constructing a straight line between the appropriate points on the scales for G_U and G_L.

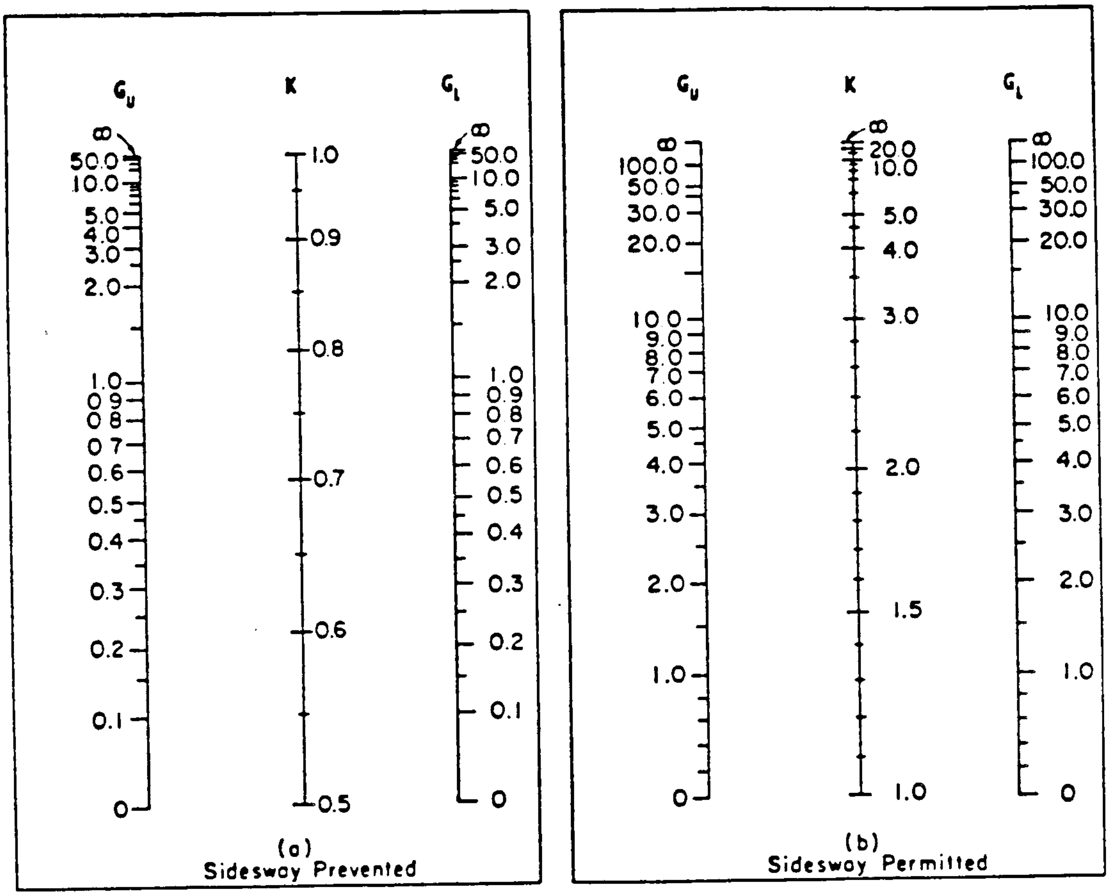


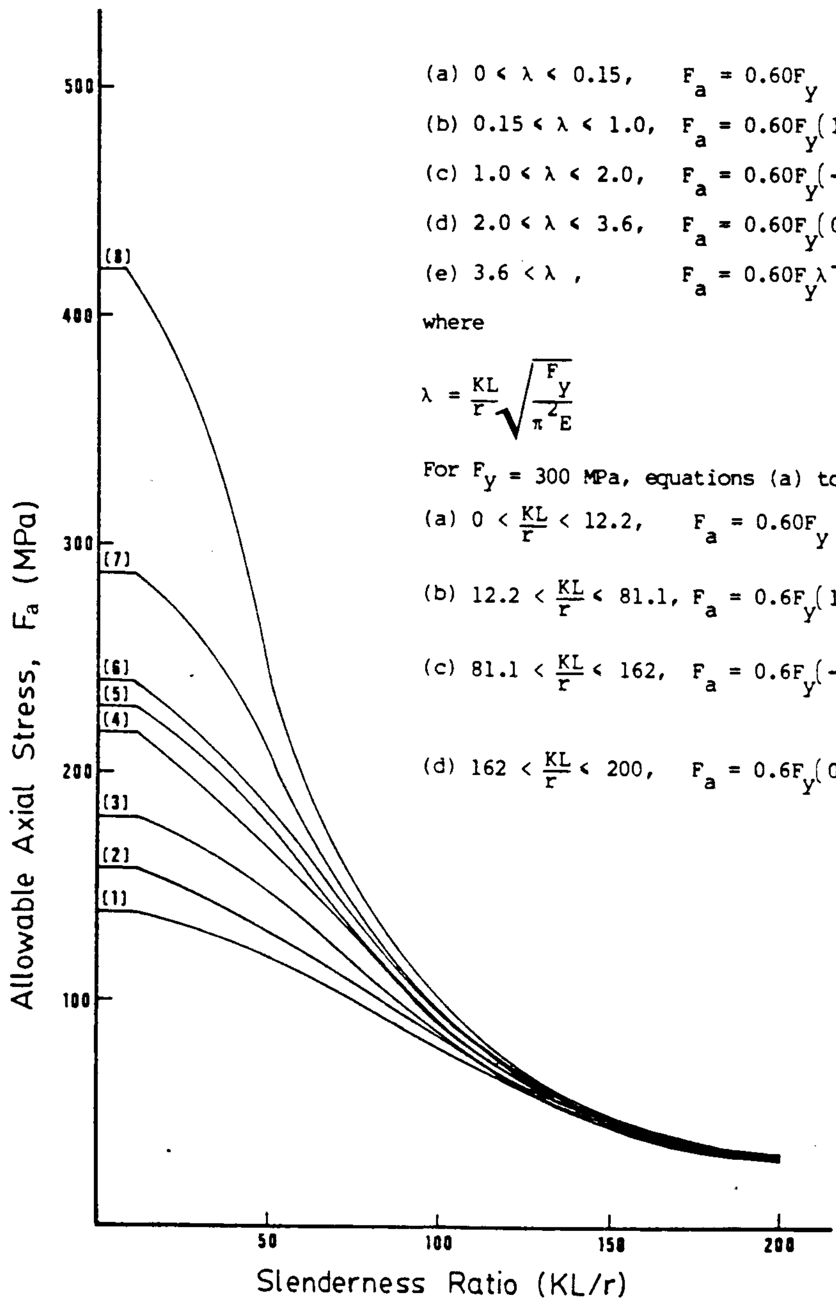
Figure C1

Alignment Chart for Effective Length
of Columns in Continuous Frames

Appendix D

Graph Showing Allowable Stress Versus Slenderness Ratio

Note: This Appendix is not a mandatory part of this Standard.



(a) $0 < \lambda < 0.15$, $F_a = 0.60F_y$

(b) $0.15 < \lambda < 1.0$, $F_a = 0.60F_y(1.035 - 0.202\lambda - 0.222\lambda^2)$

(c) $1.0 < \lambda < 2.0$, $F_a = 0.60F_y(-0.111 + 0.636\lambda^{-1} + 0.087\lambda^{-2})$

(d) $2.0 < \lambda < 3.6$, $F_a = 0.60F_y(0.009 + 0.877\lambda^{-2})$

(e) $3.6 < \lambda$, $F_a = 0.60F_y\lambda^{-2}$

where

$$\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}$$

For F_y = 300 MPa, equations (a) to (d) become

(a) $0 < \frac{KL}{r} < 12.2$, $F_a = 0.60F_y$

(b) $12.2 < \frac{KL}{r} < 81.1$, $F_a = 0.6F_y(1.035 - 0.00249\frac{KL}{r} - 0.0000337(\frac{KL}{r})^2)$

(c) $81.1 < \frac{KL}{r} < 162$, $F_a = 0.6F_y(-0.222 + \frac{51.6}{(KL/r)} + \frac{572}{(KL/r)^2})$

(d) $162 < \frac{KL}{r} < 200$, $F_a = 0.6F_y(0.009 + \frac{5770}{(KL/r)^2})$

Plot	F_y (MPa)
(1)	230
(2)	260
(3)	300
(4)	350
(5)	380
(6)	400
(7)	480
(8)	700

Figure D1

Note: For the curves plotted, equation (e) applies only to F_y = 700 Mpa with KL/r > 191.2.

Appendix E

Assumed Factors of Safety

Note: This Appendix is not a mandatory part of this Standard.

E1. The various provisions of the Standard relating to allowable stress design are based on a "factor of safety" applied to a predicable limit of structural usefulness. Such a limit may be the yield strength, the plastic strength, the tensile strength, the shear strength or the buckling strength. Buckling may occur in either the elastic or inelastic range and may be such as to affect the entire member or frame, or may be localized such as to affect only a part of a member.

E2. In plastic design, there are similar limtis of structural usefulness but in this case the margin of safety is provided by the use of a "load factor" by which the specified loads are multiplied. The factored loads must then be capable of being carried by a member or frame which has not exceeded a predictable limit of structural usefulness.

E3. Factors of safety have been established largely by experience. Those incorporated into this Standard are considered to be adequate for the usual nature and magnitude of the loads to which buildings are subject. Notwithstanding the long rcord of experience justifying the use of a basic factor of safety equal to 1.67, users of this Standard are reminded that in terms of real overload capacity the use of a single-value factor of safety for both dead loads and live loads must always lead to some inconsistency as dead loads are inherently subject to less variation. However, in working stress design there is no simple, rational means of incorporating different factors of safety for dead and live loads.

E4. The load factors 1.4 for dead load and 1.7 for live load provided for plastic design are consistent with those provided in Steel Structures for Buildings - Limit States Design when the performance factor used in that Standard to account for the variability of the member resistance is transposed by cross multiplying and incorporated in the load factors of that Standard.

E5. In the derivation of the various design provisions of this Standard the assumed factors of safety are noted below. While these may appear to vary somewhat from the basic value of 1.67, in actual construction the variation may be less than indicated because of slight deviations from the idealized situation. For example for lateral-torsional buckling of beams the assumed factor of safety is equal to 1.92. But the maximum allowable out-of-straightness may reduce the apparent factor of safety. The equations for allowable axial compression now reflect the strength reduction due to out-of-straightness and compressive residual stresses and therefore a uniform factor of safety is employed for all slenderness ratios. Bolts and other fasteners are traditionally assigned a slightly higher factor of safety in the belief that they should not be the "weakest link" in the frame or structure.

Table E1
Assumed Factors of Safety

Type of Stress	Clause	Assumed Factor of Safety
Axial tension, member and parts.	13.2	1.67 on specified yield stress at net section. 2.0 on specified tensile strength at net section. Gross section to be capable of attaining specified yield stress before net section attains specified tensile strength.
Axial tension, bolts.	13.10.2	2.50 minimum on specified tensile strength.
Axial compression, members and parts.	13.3	1.67 on predicted maximum at gross section.
Bending:	13.5	
Braced non-compact sections		1.67 on yield moment $M_y = F_y S$.
Braced compact sections		1.67 on plastic moment $M_p = F_y Z$.
Unbraced sections.		1.92 on predicted lateral-torsional buckling strength.
Shear:		
Unstiffened webs of beams	13.4	1.44 on yield shear stress taken as $0.577F_y$.
Stiffened webs of beams		1.67 on predicted shear strength combined with tension field.
Bolts.	13.10.1	2.25 minimum on predicted shear strength.

Appendix F

Guide for Floor Vibrations

Note: This Appendix is not a mandatory part of this Standard.

F1. Recent developments of floors of lighter construction, longer spans and less inherent damping have sometimes resulted in problems of objectionable floor vibrations during normal human activity. Fatique or overloading of floor structures due to vibration is not covered in the Appendix; some guidance on this is given in Reference 1 for assembly occupancies.

F2. Two types of vibration problems arise in floor construction. Continuous vibrations arise due to the periodic forces of machinery, vehicles or certain human activities such as dancing. These vibrations can be considerably amplified when the periodic forces are synchronized with a floor frequency - a condition called resonance. Transient vibrations, which decay as shown in Fig. F1, arise due to footsteps or other impact.

F3. The most important floor characteristics affecting vibration problems are the natural frequency in hertz (cycles per second) - usually that corresponding to the lowest mode of vibration - and damping. The relation between damping expressed in per cent of critical damping[2], and decay of free vibration is shown in Figure F2. Other characteristics affecting transient vibration problems are mass, especially for heavy long span floors, and stiffness under point load, especially for light short span floors.

F4. <u>**Thresholds of Annoyance**</u>

F4.1 Generally people do not like floors to vibrate. For continuous sinusoidal vibration lasting more than about ten cycles an average threshold of definite perception is shown in Figure F3 in terms of peak acceleration; the threshold levels for different people range from about one-half to twice the level shown. In the frequency range 2-8 Hz, where people are most sensitive to vibration, the threshold corresponds to 0.5 per cent g approximately, where g is the acceleration due to gravity. The threshold of definite perception shown in Figure F3 can be used to approximate a design threshold of annoyance for residential, school and office occupancies; the design level will be lower for sensitive occupancies (e.g. operating rooms, special laboratories) and greater for industrial occupancies.

F4.2 For transient vibrations, the design threshold in terms of initial peak acceleration of a decaying vibration, as shown in Figure F1, increases with an increase in damping. This is because people find continuous vibration much more annoying than vibration which quickly dies out.

F4.3 Design thresholds equivalent to that for continuous vibration are shown in Figure F3 for transient vibrations due to footsteps (walking vibrations) for different levels of damping.[3]

F5. Continuous Vibrations - Resonance

F5.1 Continuous vibrations caused by machines can be reduced by special design provisions[2,4] such as vibration isolation. Care should be taken at the planning stage to locate such machinery away from sensitive occupancies such as offices.

F5.2 Floor vibrations can also rise from heavy street traffic on bumpy pavement over soft subgrade. The annoyance increases considerably when repetitive vehicles such as buses create ground vibrations which synchronize with the floor frequency.

F5.3 Continuous vibrations caused by human activities may be a problem for light residential floors, or for long span floors used for special purposes such as dancing, concerts or gymnastics. People alone or in union can create periodic forces in the frequency range 1-4 Hz approximately, and therefore for such occupancies, natural frequencies less than 5 Hz should be avoided. To avoid very noticeable vibration for very repetitive activities such as dancing, it is recommended that the frequency of such floors be 8 Hz or more. See Reference 1 for more specific guidance on vibrations due to rhythmic human activities.

F6. Transient Vibrations

F6.1 Objectionable vibration due to footstep impact can occur in floor systems with light damping in residential, school, office and similar occupancies. Because this is the most common source of annoyance, the remainder of this guide will be concerned with this problem. Types of construction which may give transient vibration problems include open web steel joists or steel beams with concrete deck and light wood deck floors using steel joists.

F7. Performance Test for Floor Vibration

F7.1 The vibration acceptability of a floor system to human activity can be evaluated by a performance test. Partitions, rug and furnishings, finishes, etc., contribute to reduce vibration annoyance and should therefore be considered in setting up the test floor. A measuring device, which filters out frequencies greater than approximately 1.5 times the fundamental frequency, should be located near mid-span. A person who will give a subjective evaluation of the floor should also be sitting close to the measuring device.

F7.2 One test is for a person of average weight with softsoled shoes to rise up on his toes and drop on his heels near the location of measurement. Fundamental frequency, damping from the decay record (see Figure F2) and peak acceleration are obtained form the measurement and the peak acceleration is plotted on Figure F3 to see how it compares with the threshold of annoyance. Another test is to check floor comfort when different persons walk down the floor; the average peak acceleration can then be compared with the annoyance threshold for steady motion given in Figure F3.

F8. Long Span Steel Floors With Concrete Deck

F8.1 Transient vibrations may be a problem for open web steel joists or steel beams with concrete deck, composite or non-composite, generally of spans 7000-20 000 mm and frequencies in the range 4-15 Hz. For such floors, partitions, if properly located, provide more than enough damping to avoid excessive vibrations. On the other hand walking vibrations may be serious for bare floors with very low inherent damping, as is the case for fully composite construction. Figure F3 shows that the threshold of annoyance is roughly 10 times greater for 12 per cent damping than for 3 per cent damping.

F8.2 To assess vibration acceptability requires a knowledge of frequency, damping and peak acceleration from heel impact. If design by performance testing is not feasible, these parameters should be estimated by calculation as follows:

(a) **Frequency** can be estimated by assuming full composite action, even for non-composite construction. For a simply-supported one-way system, the frequency f_1 is given by:

$$f_1 = 156 \sqrt{\frac{EI_T}{wL^4}} \qquad (1)$$

where E is the modulus of elasticity of steel (200 000 MPa), I_T the moment of inertia (mm^4) of the transformed T section (concrete transformed to steel) assuming a concrete flange of width equal to the spacing of steel joists or beams, L the span in millimetres and w the dead load of the T-section in N/mm of span. Often one-way systems are supported on steel girders, and this can reduce the frequency calculated for a one-way system.

In this case the frequency can be approximated by:

$$\frac{1}{f^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2} \qquad (2)$$

where f_2 is the frequency of floor supported on steel girder perpendicular to joists. A continuous beam of equal spans on flexible supports should be treated as simply-supported since adjacent spans vibrate in opposite directions. For other conditions of span and restraint the dynamically equivalent simply-supported span is less than the full span and can be estimated from the fundamental mode shape.

(b) **Damping** is generally more difficult to estimate than frequency. A bare steel and concrete deck floor has a damping of approximately 3-4 per cent critical for non-composite construction, and about 2 per cent for fully composite construction. The addition of components such as floor finishing, rug and furnishings, ceiling, fireproofing and ducts increases the damping by about 3 per cent or more. Partitions, either above or below the floor, provide the most effective damping especially when they are located in both directions. Even light partitions which do not extend to the ceiling provide considerable damping. Partitions along with supports, or parallel to the floor joists and further apart than approximately 6000 mm, however, may not be effective because the nodal lines of vibration form under the partitions. Human beings also provide damping but this is less effective for heavy long span floors than for lighter short span floors. The following values are suggested for design calculation:

	Damping in Per Cent Critical
Bare floor	3
Finished floor - ceiling, ducts, flooring, furniture	6
Finished floor with partitions	12

(c) **Peak acceleration from heel drop** for floors greater than 7000 mm span and frequencies less than about 10 Hz, can be estimated by assuming an impulse of 70 N•s suddenly applied to a simple spring and mass system, whose mass gives the same response as that of the floor system represented as a simply-supported beam vibrating in the fundamental mode. The peak acceleration, a_o in per cent g, can be approximated by[3]:

$$a_o = \frac{(0.9)\ 2\pi f \times \text{impulse}}{\text{equivalent mass}} = \frac{60f}{wBL} \qquad (3)$$

where f is the frequency in hertz, w the weight of the floor plus contents in kPa, L the span and B the width of the equivalent beam, both in metres.

For steel joist or beam and concrete deck systems on stiff supports, L is the joist span and B can be approximated as 40 t_c, where t_c is the thickness of concrete deck determined from the average weight of concrete, including ribs. For joists or beams and concrete deck supported on flexible girders, where the girder frequency is much less than joist frequency and therefore girder vibration predominates, L is the girder span and B can be approximated as the width of floor supported by the girder. For cases where both frequencies are similar, Eqn. (2) can be used to determine frequency and BL can be estimated as follows:

$$BL = \left(\frac{f}{f_1}\right)^2 B_1\ L_1 = \left(\frac{f}{f_2}\right)^2 B_2\ L_2 \qquad (4)$$

where the subscript 1 refers to the joist or beam system on rigid supports and the subscript 2 refers to the girder system.

F8.3 For floor spans less than 7000 mm, the deflection limitations given in Clause 6.2.1.2 in this Standard are recommended, where, for non-composite construction, stiffness should be based on non-composite action. In any case, care should be taken to avoid low damping.

F9. Light Wood Deck Floors Using Steel Joists

F9.1 Transient vibrations may be objectionable for light wood deck floors using steel joists with small rolled or cold formed sections, generally with frequencies in the range 10-25 Hz. Although the same principles applying to long span floors can be used for lighter floors with higher frequencies, the motion can no longer be represented by a simple impulse applied to the floor system. This is because the persons involved - the one causing and the one receiving the motion interact with the floor to damp out the motion of the floor.

F9.2 Research carried out so far on steel joist floors with wood deck indicates that, in general, their characteristics for vibration acceptability are similar to those for wood joist floors. Evaluation tests of wood floors indicate that stiffness under point loading (approximately 1 mm maximum deflection under 1 kN) is the most important parameter affecting vibration comfort[6]. Such a stiffness requirement also helps to prevent cabinet swaying, china rattling, etc. Until research under way provides a more suitable criterion, a joist deflection limitation of L/360 under 2 kPa loading is recommended. This criterion applies only when sufficient lateral stiffness is provided either in the deck or by cross-bridging.

F9.3 Floor damping is less important for light floors than for long span floors since the main source of damping is provided by the persons on the floor. Also adding mass does not improve vibration comfort since an increase in mass corresponds to a decrease in effective damping. Spans continuous over a support which is a party wall between housing units should be avoided, since people are more annoyed by vibrations originating outside their units than from within. For cold formed C joists, ceiling boards or straps should be attached to the bottom flange to prevent annoying high frequency torsional vibrations in the joists.

F10. Corrective Measures for Unacceptable Floors

F10.1 Measures for correcting floors with annoying vibrations will depend on whether the vibrations are continuous or transient.

F10.2 For transient vibrations usually the most effective measure is to increase the damping. This can be done by adding partitions or damper posts in the floor below. If these methods are not suitable, special devices such as vibration absorbers or damping materials can be incorporated into the floor system[7,8]. For light floors a rug is effective in reducing walking impact as well as in cushioning the sway of china cabinets.

F10.3 Corrective measures for continuous vibrations include vibration isolation, smoothing of road surface and alteration of floor frequency to reduce resonance.

F11. References

(1) Supplement to the National Building Code of Canada 1985. Commentary on Serviceability Criteria for Deflections and Vibrations.

(2) Thomson, W.T. "Vibration Theory and Applications", Prentice-Hall.

(3) Allen, D.E. and H. Rainer. "Vibration Criteria for Long Span Steel Floors". Can. J. Civ. Eng., Vol. 3, No. 2, June 1976.

(4) Steffens, R.J. "Some Aspects of Structural Vibration". Building Research Current Paper Engineering Series 37, Building Research Station, Ministry of Technology, Great Britain.

(5) Allen, D.L. "Vibrational Behaviour of Long-Span Floor Slabs". Proceedings of the Canadian Structural Engineering Conference, 1974.

(6) Onysko, D.M. "Performance of Wood-Joist Floor Systems - A Literature Review". Forest Products Laboratory Information Report OP-X-24, Canadian Forestry Service, Department of Fisheries and Forestry, January 1970.

(7) Nelson, F.C. "The Use of Viscoelastic Material to Damp Vibrations in Buildings and Large Structures". AISC Engineering Journal, Vol. 5, No. 2, April 1968, p. 72.

(8) Allen, D.E. and G. Pernica "Tuned Dampers for Walking Vibrations". Proceedings of 1983 Annual CSCE Conference (Submitted to Can. J. Civ. Eng.).

(9) Lenzen, K.H. "Vibration of Steel Joists-Concrete Slab Floors". AISC Engineering Journal, Vol. 3, No. 3, July 1966, p. 133.

(10) Wright, D.T. and R. Green. "Human Sensitivity to Vibrations". Department of Civil Engineering, Report No. 7, Queen's University, February 1959.

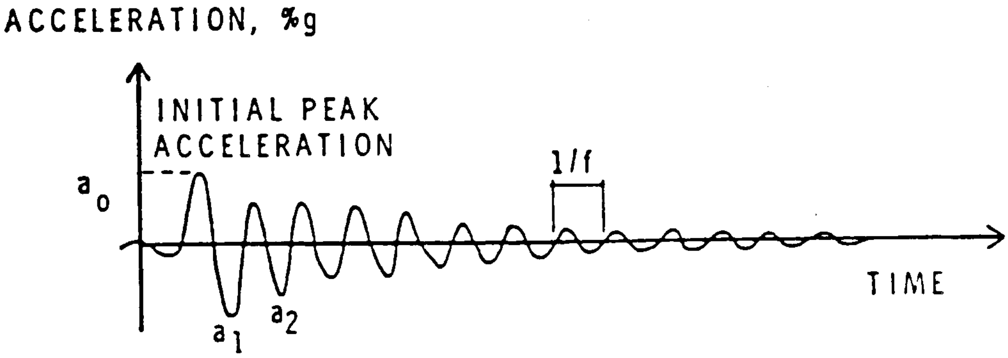


Figure F1

Typical Transient Vibration From Heel Drop
(High Frequencies Filtered Out)

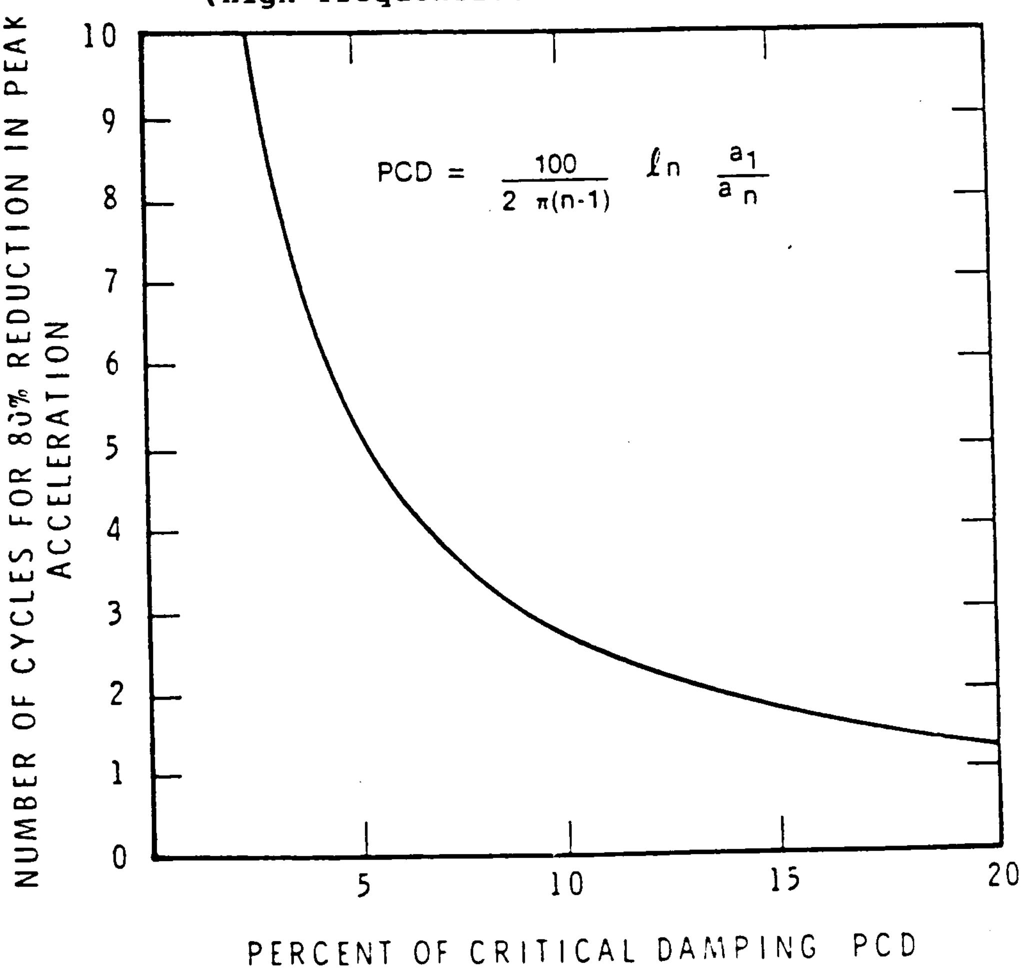


Figure F2

Relation Between Damping and Decay

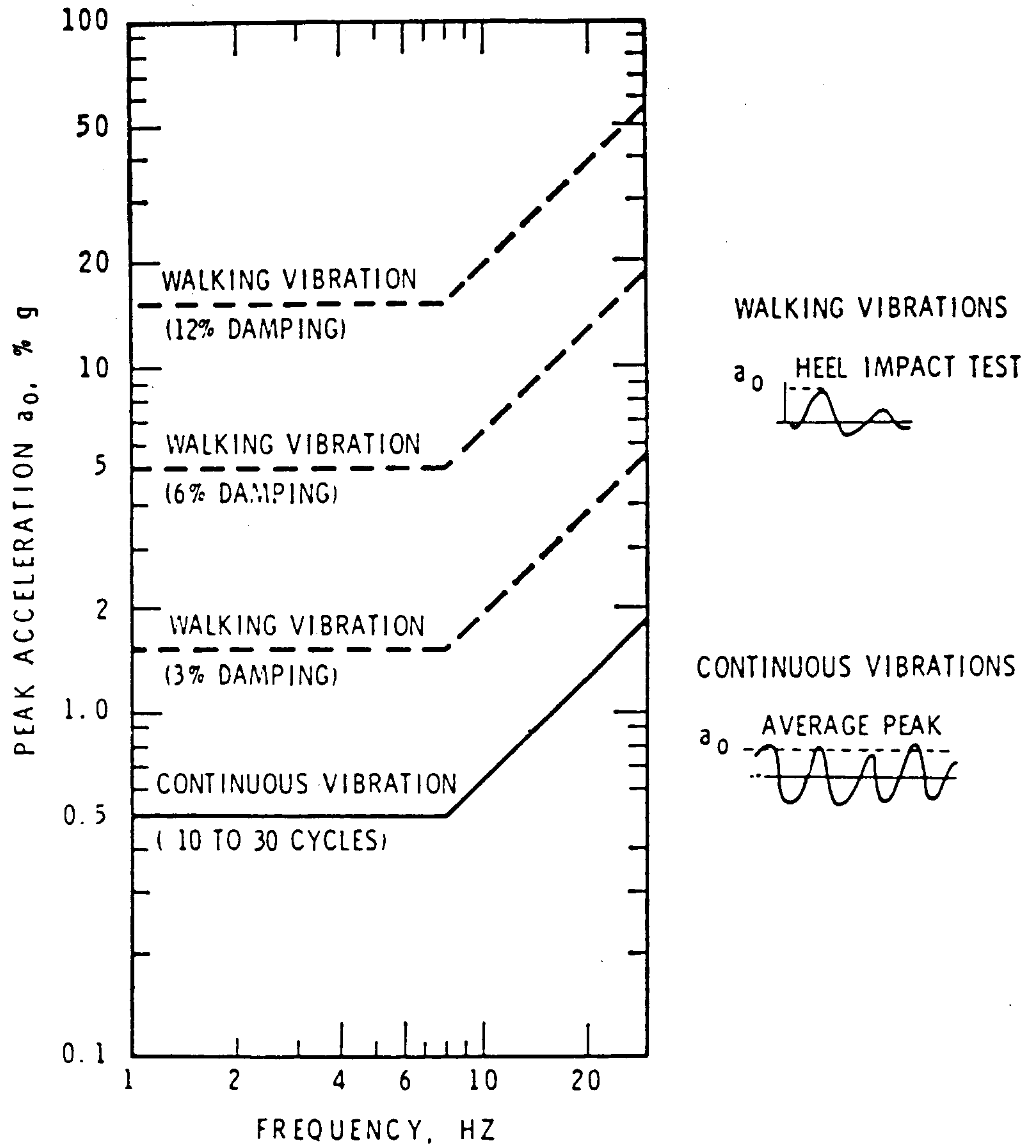


Figure F3

Annoyance Thresholds for Floor Vibrations Due to Footstep (Residential, School, Office Occupancies)

Appendix G

Wind Sway Vibrations

Note: This Appendix is not a mandatory part of this Standard.

G1. Wind motion of tall buildings or other flexible structures may create annoyance for human occupants, unless measures are taken at the design stage. The main source of annoyance is lateral acceleration, although noise (grinding and wind howl) and visual effects can also cause concern.

G2. For a given wind speed and direction, the motion of a building, which includes vibration parallel and perpendicular to the wind direction and twist, is best predicted by a wind tunnel test. Approximate calculation rules are, however, given in References 1 and 4 of Clause G4.

G3. In cases where wind motion is significant in design, the following should be considered:

(a) Education of occupants that although high winds may occasionally cause motion, the building is safe;

(b) Minimization of noises - detailing of building joints to avoid grinding, design of elevator guides to avoid scraping due to sway;

(c) Minimization of twist by symmetry of layout, bracing or outer walls (tube concept). Twist vibration also creates a magnified visual effect of relative motion of adjacent buildings;

(d) Possible introduction of mechanical damping to reduce wind vibration.

G4. **References**

(1) Supplement No. 4 to the National Building Code of Canada, 1977, Commentary on Wind Loads.

(2) Hansen, R.J., Reed, J.W. and Van Marcke, E.H. Human Response to Wind-Induced Motion of Buildings, Journal of the Structural Division, ASCE, Vol. 99, No. ST7, July 1973, p. 1589-1605.

(3) Chen, P.W. and Robertson, L.E. Human Perception Thresholds of Horizontal Motion. Journal of the Structural Division, ASCE, Vol. 98, No. ST8, August 1972, p. 1681-1695.

(4) Reed, J.W. Wind-Induced Motion and Human Discomfort in Tall Buildings. Department of Civil Engineering Research Report R71-42. Massachusetts Institute of Technology, November 1971.

(5) Hogan, M. The Influence of Wind on Tall Building Design. Faculty of Engineering Science Research Report BLWT-4-71, University of Western Ontario, March 1971.

(6) Council on Tall Buildings and Urban Habitat. Monograph on the Planning and Design of Tall Buildings. Volumes PC and SB. American Society of Civil Engineers 1981.

Appendix H

Recommended Maximum Values for Deflections

Note: This Appendix is not a mandatory part of this Standard.

For Specified Design Live and Wind Loads*

		Due to:		
INDUSTRIAL TYPE BUILDINGS	Vertical Deflection	Live Load	Simple span members supporting inelastic roof coverings	$\frac{1}{240}$ of span
		Live Load	Simple span members supporting elastic roof coverings	$\frac{1}{180}$ of span
		Live Load	Simple span members supporting floors	$\frac{1}{300}$ of span
		Maximum Wheel Loads (no impact)	Simple span crane runway girders for crane capacity of 225 kN and over	$\frac{1}{800}$ of span
		Maximum Wheel Loads (no impact)	Simple span crane runway girders for crane capacity under 225 kN	$\frac{1}{600}$ of span
	Lateral Deflection	Crane Lateral Force	Simple span crane runway girders	$\frac{1}{600}$ of span
		Crane Lateral Force OR Wind	Building column sway†	$\frac{1}{400}$ to $\frac{1}{200}$ of height

ALL OTHER BUILDINGS	Vertical Deflection	Live Load	Simple span members of floors and roofs supporting construction and finishes susceptible to cracking...............	$\frac{1}{360}$ of span
		Live Load	Simple span members of floors and roofs supporting construction and finishes not susceptible to cracking...............	$\frac{1}{300}$ of span
	Lateral Deflection	Wind	Building sway, due to all effects...............................	$\frac{1}{400}$ of bldg. height
		Wind	Storey drift, (relative horizontal movement of any two consecutive floors due to the shear effects) in buildings with cladding and partitions without special provision to accommodate building frame deformation..........	$\frac{1}{500}$ of storey height
		Wind	The same, with such special provision..........................	$\frac{1}{400}$ of height

* Although this Appendix refers specifically to Specified Design Wind and Live Loads when setting forth deflection criteria, the designer should consider the inclusion of Specified Dead Loads in some instances. For example, non-permanent partitions, which are classified by the National Building Code as dead load, should be part of the loading considered under Appendix H if they are likely to be applied to the structure after the completion of finishes susceptible to cracking. Because some building materials augment the rigidity provided by the steelwork, the wind load assumed carried by the steelwork, for calculating deflections can be somewhat reduced from the design wind used in strength and stability calculations. The more common structural elements contributing to the stiffness of a building are masonry walls, certain types of curtain walls, masonry partitions and concrete around steel members. The **maximum** suggested amount of this reduction is 15 per cent. In tall and slender structures (height greater than **4 times** the width) it is recommended that the wind effects be determined by means of dynamic analysis, or wind tunnel tests.

† Permissible sway of industrial buildings varies considerably depending on factors such as wall construction, building height, effect of deflection on the operation of crane, etc. Where the operation of the crane is sensitive to the lateral deflections, a permissible lateral deflection less than 1/400 of the height may be required.

Appendix I

Guide to Calculation of Stability Effects

Note: This Appendix is not a mandatory part of this Standard.

I1. General

I1.1 This Appendix gives one approach to the calculation of the additional bending moments and forces generated by the vertical loads acting through the deflected shape of the structure. By this approach the above moments and forces are incorporated into the results of the analysis of the structure; alternatively a second order analysis, which formulates equilibrium on the deformed structure, may be used to include the stability effects.

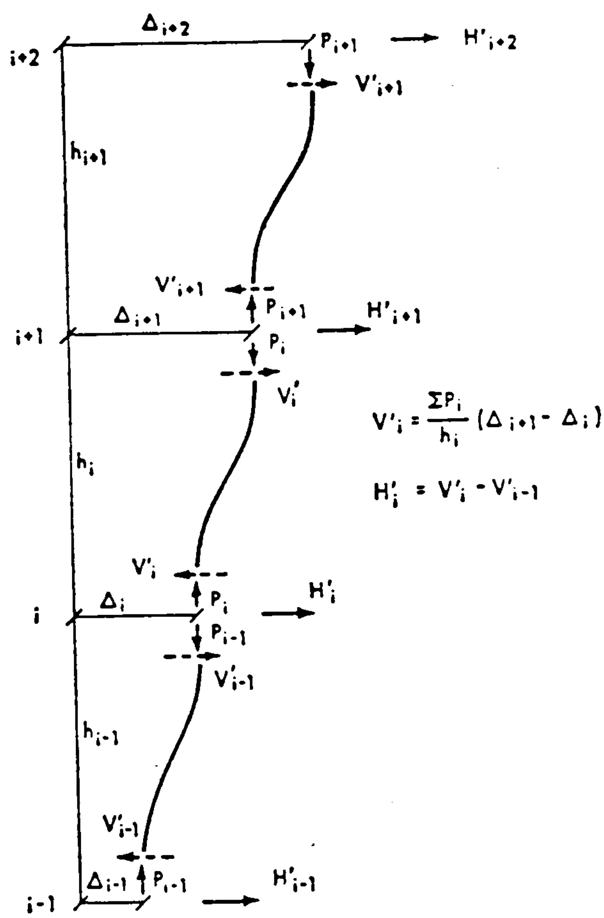


Figure I1

Sway Forces Due to Vertical Loads

I2. **Combined Loading Case**

I2.1 Step 1 - Apply the vertical and lateral loads (multiplied by the appropriate load combination factor) to the structure (see Clause 7.2.2).

Step 2 - Compute the lateral deflections at each floor level (Δ_i) by first order elastic analysis

Step 3 - Compute the artificial storey shears V_i' due to the sway forces.

where

V_i' $= 1.7 \frac{\Sigma P_i}{h_i} (\Delta_{i+1} - \Delta_i)$

= artificial shear in storey i due to the sway forces

ΣP_i = sum of the column axial loads in storey i

h_i = height of storey i

Δ_{i+1}, Δ_i = displacements of level i + 1 and i, respectively

Step 4 - Compute the artifical lateral loads H_i'

$$H_i' = V_{i-1}' - V_i'$$

Step 5 - Repeat Step 1 applying the artificial lateral loads H_i' in addition to the applied loads.

Step 6 - Repeat Steps 2 through 5 until satisfactory convergence is achieved. Lack of convergence within 5 cycles may indicate an excessively flexible structure.

I3. **Vertical Loads Only**

I3.1 Because vertical loads do not normally produce significant sway deflections of the structure the initial sway forces are computed on the basis of the sway displacements in each storey equal to the erection tolerance permitted by Clause 28.7.1. Using these deflections the calculations are commenced at Step 3 of the procedure described in Clause I2.1.

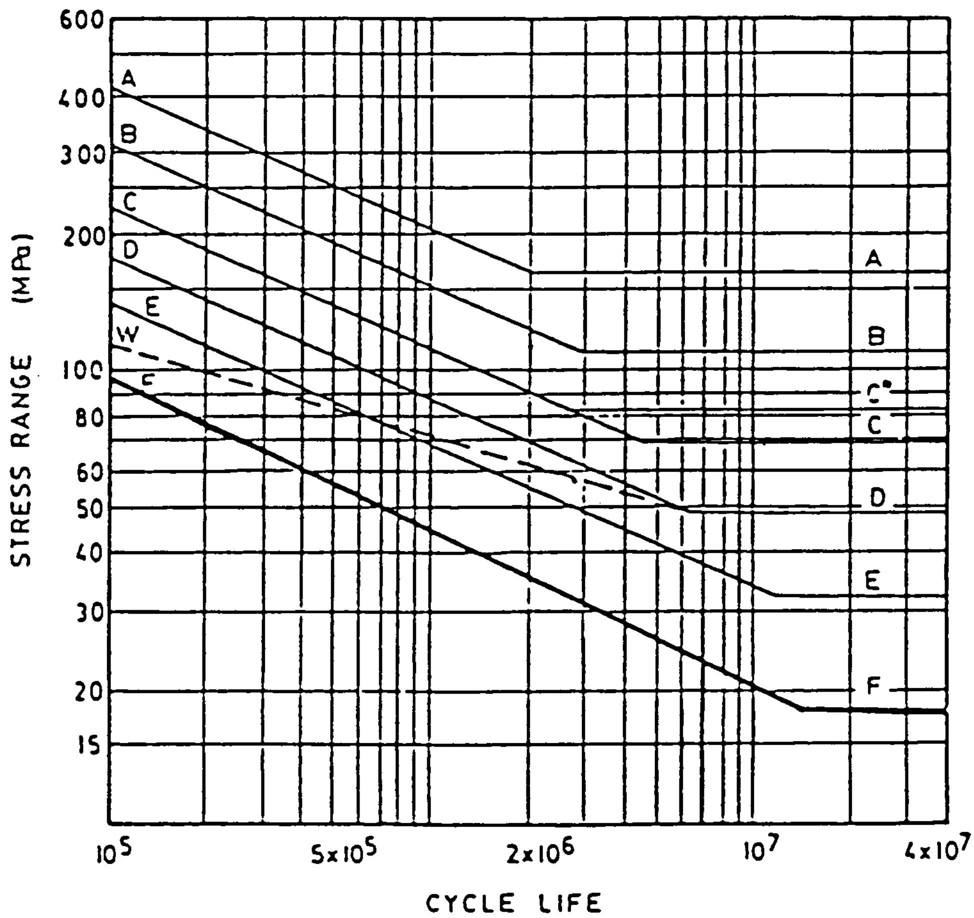
Appendix J

Fatigue

Note: This Appendix is not a mandatory part of ths Standard.

J1. Figure J1 is a plot of the design curves for the allowable stress range for categories A to F of Tables 11(a) and (b).

J2. Figure J2 gives illustrative examples of the various fatigue categories described in Table 11(b).



**Except for transverse stiffener welds on girder webs or flanges where 83 M Pa should be used.*

Figure J1

Design Curves for the Allowable Stress Range for Categories A to F

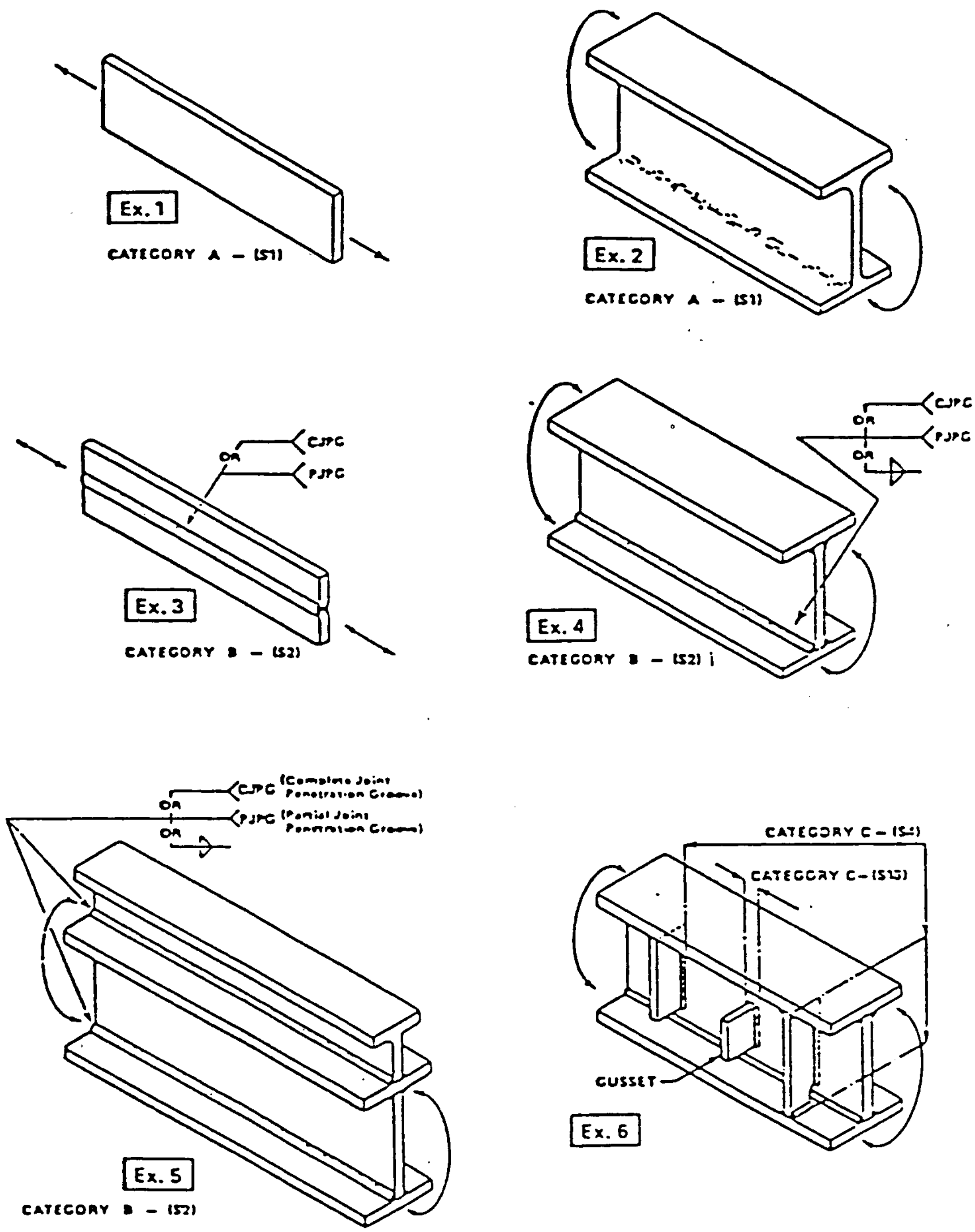


Figure J2
Illustrative Examples of Various Details
Representing Stress Range Categories

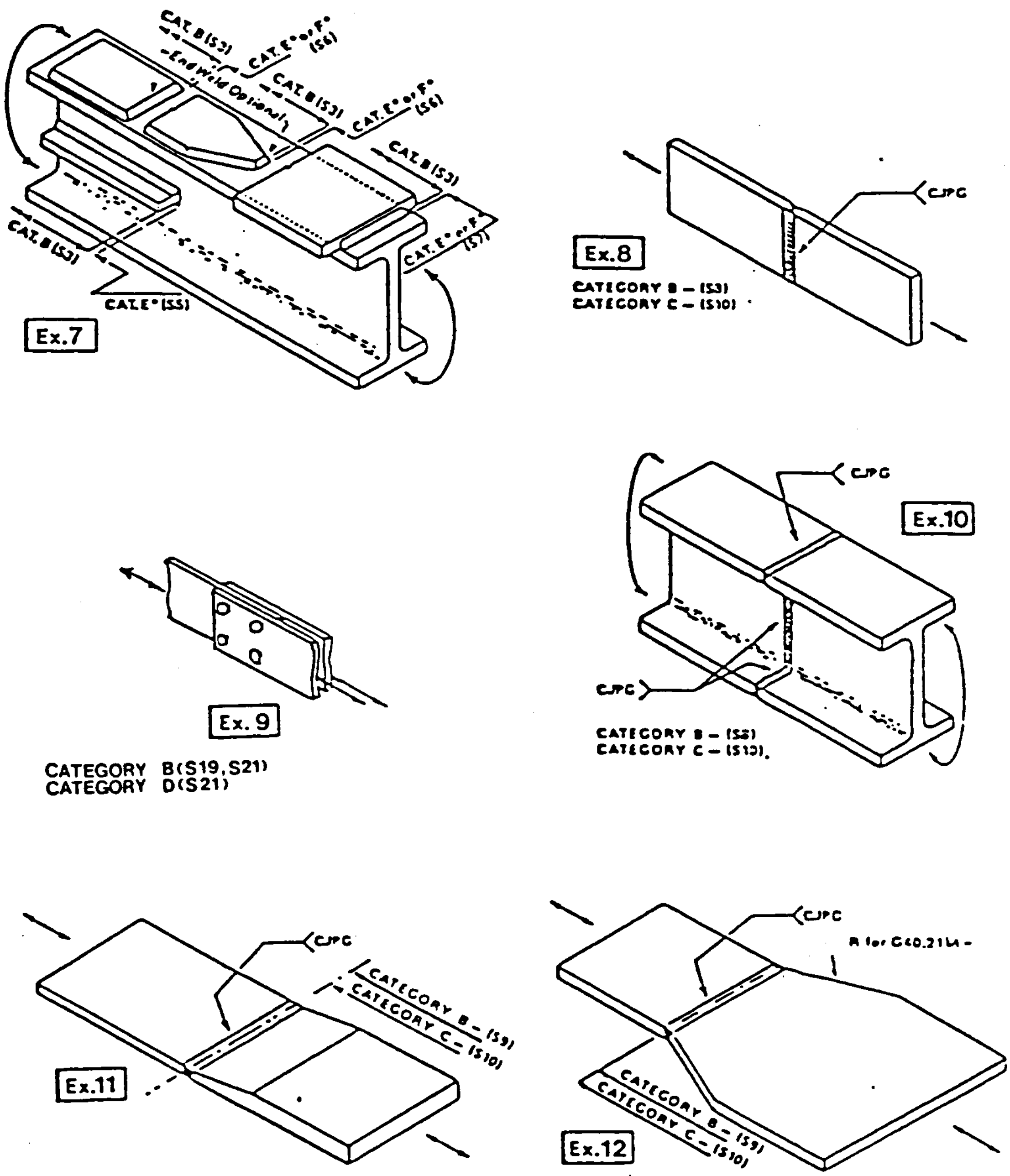
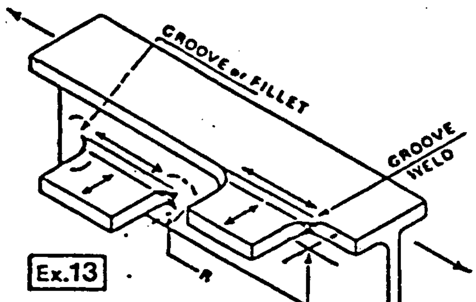


Figure J2
Illustrative Examples of Various Details
Representing Stress Range Categories (cont'd)



Ex.13

(S11, .S15)
Applicable Categories and Governing Conditions are shown at right.

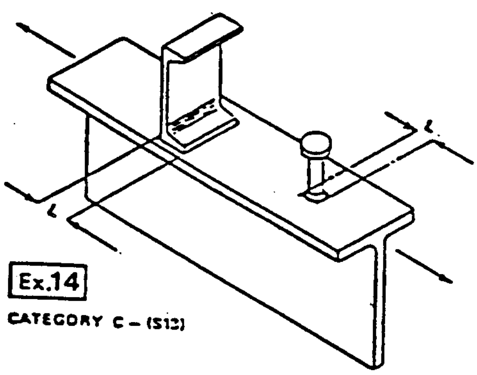
Ex. 13	Fillet Connections	Groove Connections				
	To Web	To Web To Flange	To Web (3)	To Flange (3)		
	Longitudinal Loading/Transverse Loading	Longitudinal Loading	Transverse Loading			
Transition Radius "R" (millimetres)	Stress Range Category		Stress Range Category	Stress Range Category based on Condition of Joint (2)		
				1	2	3,4
50 > R > 0	E	E(1)	E	E	E	E
150 > R > 50(4)	D	D	D	D	D	E
600 > R > 150(4)	D	C	C	C	C	E
R > 600(4)	E	B	C	B	C	E

1. For longitudinal loading only, use Category D if detail length is between 50 mm and 12 times the plate thickness, but less than 100 mm.

2. Condition of Joint:
   (1) Equal thickness of parts joined - reinforcement removed.
   (2) Equal thickness of parts joined - reinforcement not removed.
   (3) Unequal thickness of parts joined - reinforcement removed.
   (4) Unequal thickness of parts joined - reinforcement not removed.

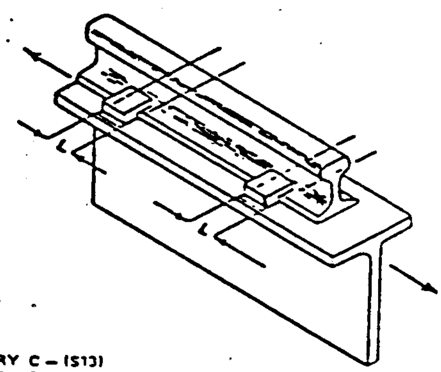
3. Weld soundness to be established by nondestructive examination.

4. Terminal ends of welded joints to be ground smooth.



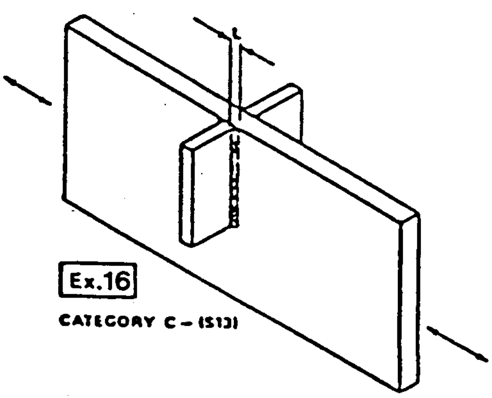
Ex.14

CATEGORY C – (S13)



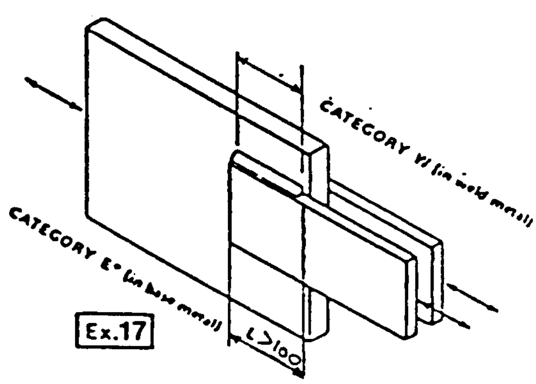
Ex.15

CATEGORY C – (S13)
CATEGORY D – (S14)



Ex.16

CATEGORY C – (S13)



Ex.17

Figure J2
Illustrative Examples of Various Details
Representing Stress Range Categories (cont'd)

Appendix K

Deflections of Composite Beams Due to Shrinkage Strain

Note: This Appendix is not a mandatory part of this Standard.

K1.1 During the curing of the concrete cover slab, the volumetric reduction (shrinkage strain) of the concrete induces additional flexural deflections in the composite member when the concrete slab contracts but the steel shape does not.

K1.2 Two models have been suggested[1,2] to estimate the deflection due to shrinkage. Basically, these models assume a value of shrinkage strain acting on the effective area of the concrete slab. It is assumed that the shrinkage strain is equivalent to a constant moment acting on the composite beam by selecting an appropriate modulus of elasticity for concrete and a lever arm of the force taken from the centre of the slab to the elastic neutral axis of the composite beam (Figure K.1).

K2. The resulting shrinkage deflection is:

$$\Delta_{sh} = \frac{\varepsilon_{sh} E_c A_c L^2}{8EI} \cdot y_c$$

where

ε_{sh} = the shrinkage strain
E_c = modulus of elasticity of concrete
A_c = effective area of concrete slab
L = span length of beam
E = modulus of elasticity of steel beam
I = moment of inertia of composite beam as given in K3 or K4
y_c = distance from elastic neutral axis to assumed line of action of the shrinkage face.

K3. The first method[1] uses the restrained shrinkage value for ε_{sh}, which is lower than the free shrinkage strain, the normal modulus of elasticity, E_c, and the transformed moment of inertia, I_t.

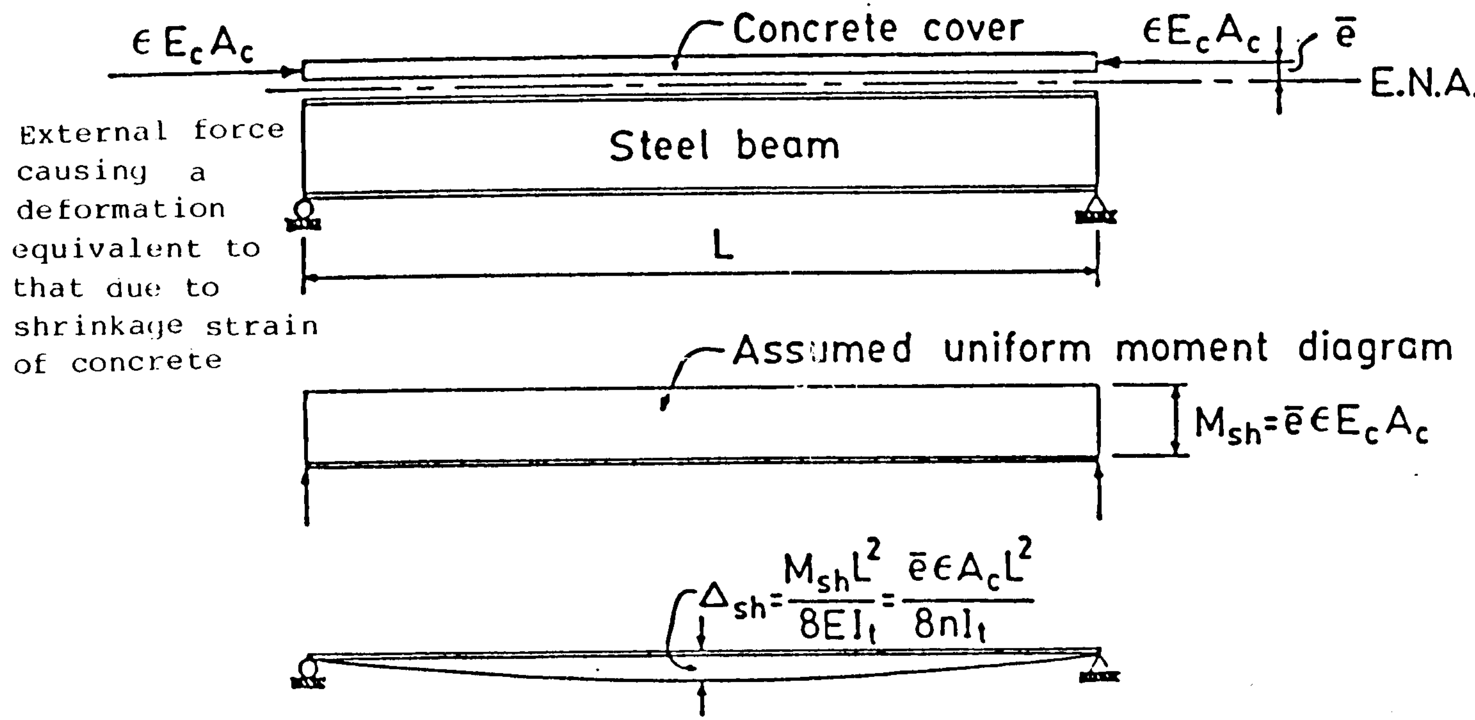
K4. The second method[2] takes the shrinkage strain as the strain appropriate to free shrinkage and reflects the influence on the free shrinkage of the following: the time over which shrinkage occurs, relative humidity, volume-to-surface ratio, slump, fires, air contact and cement contact of the concrete mix. An age-adjusted modulus, similar to that used in calculations of concrete creep, is used for E_c and the effective moment of inertia I_e.

K5. For both methods, care should be taken in selecting values for the quantities in equation K.1 so as to neither over or under estimate deflections especially when used to determine the serviceability requirements. Many of the quantities in equation K.1 will be influenced by site conditions.

K6. **References:**

(1) Chien, E.Y.L., "Composite Floor Systems" (to be published by Canadian Institute of Steel Construction).

(2) Montgomery, C.J., Kulak, G.L., and Shwartsburd, G., "Deflection of a Composite Floor System", Can. J. Civ. Eng., Vol. 10, No. 2, June, 1983.



ϵ E_c A_c
Concrete cover
ϵE_c A_c
ē
E.N.A.
External force causing a deformation equivalent to that due to shrinkage strain of concrete
Steel beam
L
Assumed uniform moment diagram
M_sh = ē ϵ E_c A_c
Δ_sh = M_sh L² / 8EI_t = ē ϵ A_c L² / 8nI_t

Figure K1

FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

$Feet^2$ (ft^2) x 0.0929030	= $Meters^2$ (m^2)
Acres x 4046.86	= $Meters^2$ (m^2)
$Miles^2$ (mi^2) x 2.58999	= $Kilometers^2$ (km^2)

VOLUME

$Feet^3$ (ft^3) x 0.02831685	= $Meters^3$ (m^3)
$Feet^3$ (ft^3) x 28.31685	= Liters**
$Yards^3$ (yd^3) x 0.764555	= $Meters^3$ (m^3)
Acre feet x 1233.48	= $Meters^3$ (m^3)
Gallons (gal) x 0.00454609	= $Meters^3$ (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

$Feet^3$ per second (ft^3/s) x 0.02831685	= $Meters^3$ per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= $Meters^3$ per second (m^3/s)
Acre-feet per day x 0.0142764	= $Meters^3$ per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= $Meters^3$ per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 (lb-s/ft^2) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

---

* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

NOTES

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 7C

Structural Design Requirements
STRUCTURAL STEEL

COMMENTARY

1985

PART 2

SECTION 7

TABLE OF CONTENTS

FOREWORD

This Section is divided into three parts:

Section 7A - Limit States Design
Section 7B - Working Stress Design
Section 7C - Commentary

The work on this Section was carried out by Messrs. Adams, Kennedy and Kulak of the Department of Civil Engineering, University of Alberta, under a contract with the Caribbean Community. It is suggested that comments on the alternative design methods, or on any of the design details recommended, be sent to the authors.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

Each of the Sections 7A, Limit States Design; 7B, Working Stress Design; and 7C, Commentary have been numbered individually so as to provide continuity between sections. The number and digit corresponding to the Part and Section in the Part (2.7) have been omitted. The numbers that remain correspond to the sub-sections and articles.

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 7C

STRUCTURAL STEEL

COMMENTARY

Disclaimer

The authors of this publication, Messrs. Adams, Kennedy and Kulak, do not warrant the contents and the suitability of this material for general or particular use, nor is there any implication that there is freedom from infringement of patents. The design of structures is within the scope of expertise of a competent licensed professional. Such user of this Standard assumes all liability arising from its use. Although every effort has been made in writing and proofreading this Standard to ensure that all information is accurate and that all numerical values are correct some errors may have been overlooked. Users are requested to bring any such errors found to the attention of the authors.

Preface

This commentary has been prepared to clarify the intent of various provisions of Caribbean Uniform Building Code Project "Structural Steel for Buildings", Part 2.7A - Limit States Design and Part 2.7B - Working Stress Design. The publications listed as references provide an extensive background to the development of the Standard and its technical requirements.

Although every effort has been made in writing and proofreading this Standard to ensure that all information is accurate and that all numerical values are correct some errors may have been overlooked. Users are requested to bring any such errors found to the attention of the authors.

Introduction

Limit States Design

2.7A - Limit States Design of Steel Structures for Buildings is based on the design philosophy whereby the designer proportions the structure such that various limit states are not exceeded. Limit states are those limiting states or conditions of a structure at which it ceases to fulfil its intended function. The conditions associated with collapse of all or part of the structure and including rupture, crushing, buckling, local buckling, critical moment, yield moment, plastic moment, mechanism formation, overturning, sliding and foundation failure are called ultimate limit states. The conditions associated with providing proper acceptable service conditions such as deflections, vibrations, cracking, crack growth, permanent deformations and settlement are called serviceability limit states.

In limit states design, the capacity or performance of the structure or its components is checked against the various limit states at certain load levels. For the ultimate limit states of strength and stability, the structure must retain its load carrying capacity at the factored load levels. For the serviceability limit states, the performance of the structure at specified load levels must be satisfactory. The specified loads are those prescribed by the Regulatory Authority and a factored load is the product of a specified load and its load factor.

The limit states design standard uses partial factors, a load factor and a resistance factor, derived statistically.

A load factor (α) is applied to a specified load to take into account the fact that loads higher than those anticipated may exist and also to take into account approximations in the analysis of the effects of the load. A resistance factor (ϕ) is applied to the nominal member strengths, or resistances (R), to take into account that the resistance of the member, due to variability of the material properties, dimensions and workmanship, may differ from that anticipated and also to take into account the type of failure and uncertainty in the prediction. An advantage, therefore, of limit states design is that the factors assigned to loads arising from different sources can be related to their uncertainty of prediction of the resistance, and the factors assigned to different members can be related to their reliability and to the different types of failure. Thus, a greater degree of consistency against failure can be obtained[1,2,3].

For failure of structural steel members by yielding, the resistance factor is taken to be 0.90[112]. To maintain simplicity in design, the resistance formulas for buckling or other types of member failure have been adjusted so that a uniform resistance factor, ϕ = 0.90, can be used and yet provide the necessary safety. For example, the resistance formula for tension, LSD Clause 13.2(a)(i), provides a higher safety factor for members which fracture across the net section before the member can yield in the gross section. The only exceptions to a value of 0.90 for the resistance factor are:

(1) Bolts in bearing type connections and welds, where ϕ = 0.67, to ensure that the bolts and welds will be stronger than the members being joined;

(2) Crushing resistance of concrete (for composite construction), where ϕ_c = 0.60 to take into account the greater strength variability and type of failure associated with concrete.

(3) Shear connectors (for composite construction), ϕ_{sc} = 0.80.

Probabilistic studies[2] show that consistent probabilities of failure are determined for all dead-to-live ratios when a dead load factor of 1.25 and a live load factor of 1.50 are used. The live load factor of 1.50 when divided by the performance factor of 0.90, gives a factor of 1.67 which is the nominal factor of safety in 2.7B - Working Stress Design.

Working Stress Design

2.7B - Working Stress Design of Steel Structures for Buildings, is based on working stress and, with the exception of the clauses on plastic design, requires the calculation of stresses at various points in the structure and a comparison of these stresses with allowable stresses. The allowable stresses are usually established as some portion of the yield point of the material. Elements or members subjected to compression are examined for stability, usually expressed as an allowable compressive stress. The working stress equations are generally similiar to the equations expressing member resistances in Part 1 - Limit States Design and provide a nominal value of the factor of safety against ultimate capacity, whether determined by yielding of the cross section or by instability, of 1.67. The factor of 1.67 is equal to the live load factor of 1.50 in the Limit States Design part divided by the resistance factor of 0.90.

Comparison of the Two Design Methods

Figure 1 compares LSD with WDS for beams. The safety index, β, which is a measure of the probability of failure is plotted in the upper part of the figure. It is seen that the LSD gives more uniform safety, $2.9 < \beta < 3.3$, than does WDS, $3.3 < \beta < 3.7$. The ratio of the resistances required by the two standards is plotted in the lower part of the figure. The open circle identifies the case when both standards give the same required resistance.

Figure 2 shows the variation in the safety index, β, for WSD and LSD members in axial compression for the two loading cases indicated. For LSD the safety index ranges from 3.0 to 3.5 while for WSD it ranges from 3.2 to 3.9. Note that the column formula produces a consistent safety level for short and

Figure 1

COMPARISON OF LSD AND WSD — BEAMS

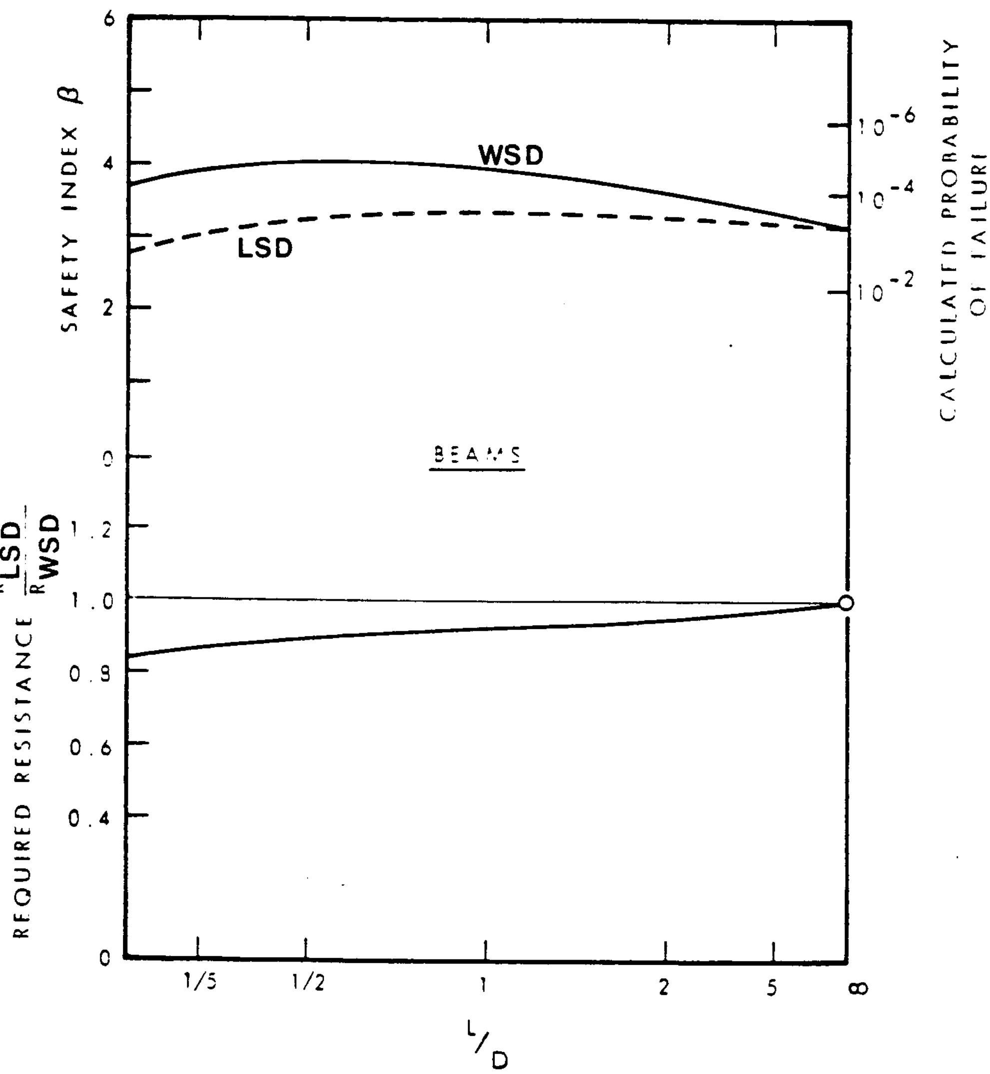
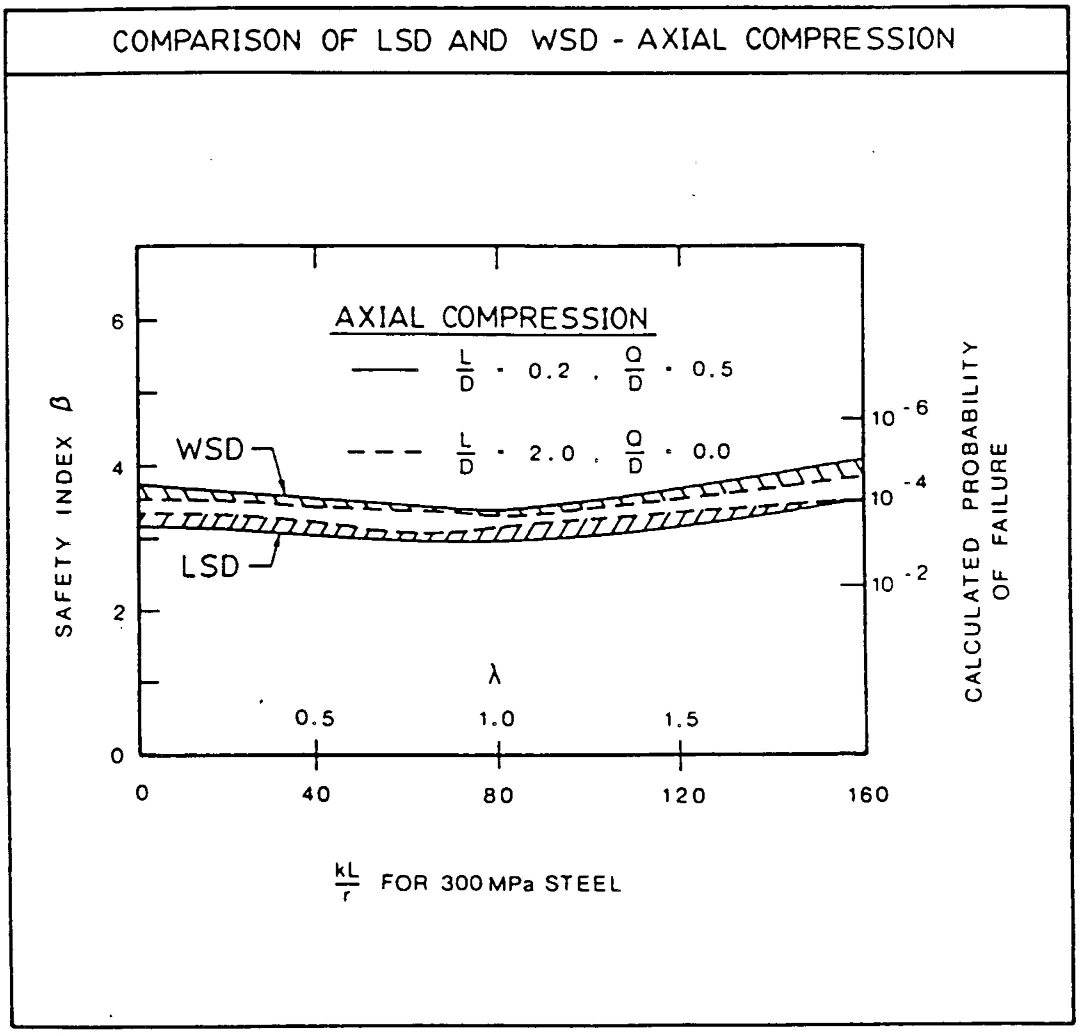


Figure 2



intermediate columns, and a little more safety for very long columns.

The LSD safety indices lie closer to the target value of 3.0 which has produced satisfactory reliability for beams and columns. Therefore the required safety is being obtained with greater economy. Overall a material saving of roughly 7 per cent is expected.

The saving in material is essentially due to a reduction in the load factor for dead load compared to previous standards. The justification for this is that dead loads can be predicted more closely both in magnitude and distribution and can be controlled more closely than the variable loads.

References 1 and 2 provide considerably more information on the type of probabilistic, calibration and design studies that were performed while developing the limit states standard. Reference 4 contains a more extensive discussion on limit states design in the National Building Code of Canada. Reference 112 provides information on the statistical determination of the resistance factor (ϕ).

In the Commentary clauses that follow, the numbers and headings used refer to the relevant clause numbers and headings of both Parts, 2.7A - Limit States Design and 2.7B - Working Stress Design. For simplicity 2.7A - Limit States Design will be referred to as LSD and 2.7B - Working Stress Design will be referred to as WSD. For many of the clauses the commentary is common to both the LSD and WSD parts. Where necessary comments on the two separate parts are given.

1. SCOPE

When structures are designed using 2.7A - Limit States Design, no use shall be made of 2.7B - Working Stress Design or vice versa. In 2.7B - Working Stress Design only specified loads are used for determining the strength of members, while in 2.7A - Limit States Design, both specified and factored loads are used. Thus, by staying within one standard the possibility of confusion as to which loads are being considered is reduced.

2. APPLICATION

Clause 2.3 of this section notes that the designer has the freedom (subject to approval from the Regulatory Authority) to use methods of design or analyses in lieu of the formulae given in the Standards. It is required, of course, that the same margins of safety implicit in the Standards be provided by the alternate methods.

5. MATERIALS: STANDARDS AND IDENTIFICATION

The design requirements contained in 2.7A and 2.7B have been developed on the assumption that the materials and products which will be used are those listed in Clause 5. These materials and products are all covered by standards prepared by the Canadian Standards Association (CSA) or the American Society for Testing and Materials (ASTM).

The standards listed provide controls over manufacture and delivery of the materials and products which are necessary to ensure that the materials and products will have the characteristics assumed when the design provisions were prepared. The use of materials and products other than those listed is permitted, but the designer should assure himself, when this option is used, that the materials and products have the characteristics required to perform satisfactorily in the structure. In particular, ductility is often as important as the strength of the material. Weldability and toughness may also be required in many structures.

6. DESIGN REQUIREMENTS

Many of the serviceability requirements (deflections, vibrations, etc.) are stipulated qualitatively and guidance, in quantitative form is provided in Appendices. Thus, the designer is permitted to use the best information available to him in order to satisfy the serviceability requirements, but is also provided with information that is considered to be generally suitable, when used with competent engineering judgement.

6.2.2 (LSD), 6.2.1 (WSD) The basic requirement of this clause is that deflection of both members and frames under the influence of design loads must be kept within acceptable limits for the nature of the supported materials and the intended use and occupancy.

6.2.3 (LSD), 6.2.2 (WSD) If a flat roof is too flexible, rainwater will not accumulate evenly over the roof but will flow to form ponds in a few local areas. This may lead to an instability condition similar to buckling which can result in failure of the roof due to local overloading.

It is considered good practice to take into account, when determining the location of roof drains, not only the roof slope but also deflection of the roof due to creep and rain. Drains should be provided with suitable devices to prevent clogging by leaves or, when appropriate, suitable overflows should be provided through parapet walls.

6.2.4 (LSD), 6.2.3 (WSD) Camber is usually provided to offset dead-load deflections so that the finished, but otherwise unloaded, floor or roof is theoretically flat. In some cases an arching action occurs and the camber does not reduce in the amount anticipated. Where this upward curvature cannot be tolerated the designer should be cautious in specifying camber requirements and be guided by experience with previous similar construction.

In Part 1 - Limit States Design Clause 6 clearly distinguishes between those requirements which must be checked using specified loads (serviceability limit states) and those which must be checked using factored loads (ultimate limit states).

6.3.3 The philosophy of designing buildings against earthquakes is that they should suffer no significant damage when subject to moderate earthquakes and that they should not collapse when subject to major earthquakes[12]. If collapse is prevented the occupants can exit from the building. As ground disturbances due to active faulting, slides or subsidence cannot be predicted the provisions of this Standard do not guard against them.

The designer should consider special design criteria for unusual, irregular or special purpose structures.

Steel structures, to satisfy the above philosophy, must have sufficient ductility to allow the development of plastic hinges and the redistribution of bending moments from one part of the structure to another (see References 122 and 123). Therefore, the structure must meet the requirements for structures analyzed plastically. To limit the $P\Delta$ effects, that is the disturbing effect of the vertical loads acting on the structure which has swayed sideways due to the earthquake, a limit of 0.03 of the storey height is placed on the storey drift[124].

In some design situations under major earthquake loading beam webs will be required to yield cyclically. It is suggested that the requirements for h/w limits given in Reference 125, which may be more severe than those for class 1 sections be met to ensure that local buckling does not occur.

7. LOADS AND SAFETY CRITERION

Limit States Design

This clause sets forth the fundamental safety criterion that must be met, namely:

Factored Resistance $\geq$ Effect of Factored Loads

The expression for the effect of factored loads is:

$$\alpha_D D + \gamma\ \psi\ (\alpha_L L + \alpha_Q Q + \alpha_T T)$$

The Factored Resistance is given by the product ϕ R, where ϕ

is the resistance factor discussed in the introduction to this commentary and R is the nominal member strength, or resistance. The factored resistances of various types of members are given in Clauses 13, 16, 17, 18 and 21.

Working Stress Design

This clause sets forth the fundamental safety criterion that must be met:

Allowable Stress ≥ Stress due to the Specified Loads

The expression for the Effect of Specified Loads is:

$$\psi\ [D - \gamma\ (L + Q + T)]$$

The allowable stresses do not depend on the load combination and the loads, other than dead load, are multiplied by a load combination factor that depends on the number of loads being considered.

8. ANALYSIS OF STRUCTURE

Clause 8 permits the use of two basic types of construction- "continuous" and "simple" that are defined there. In recognition of previous successful practice, a special form of "simple" construction is permitted. In this form of construction, a building frame may be designed to support gravity loads on the basis of "simple" construction and to support lateral loads due to wind or earthquake through the provision of moment-resisting joints. A number of limitations are imposed in Clause 8.3.2 if this method is used.

The limitations are intended to ensure that the moment-resisting joints designed nominally for wind or earthquake moments alone have both the strength and ductility necessary to accommodate the "overload" which will result if factored (LSD) or full (WSD) gravity and lateral loads act concurrently. It is assumed that, if the connection has adequate capacity for inelastic rotation when subjected to the first application of factored (LSD) or full (WSD) gravity and lateral loading, under subsequent loading cycles the connection will behave elastically although it will have a permanent inelastic deformation[9,10]. Such an assumption is valid except in joints where load fluctuation would create alternating plasticity in the connection[11].

Clause 8 also permits the use of two general methods of analysis-elastic and plastic. Methods of elastic analysis are familiar to most designers. A brief explanation of the plastic analysis requirements of Clause 8.5 follows.

8.5 Plastic Analysis

The use of plastic analysis at the factored load levels to determine the forces and moments throughout a structure implies that the structure achieves its limit of usefulness when sufficient plastic hinges have been developed to transform the frame into a mechanism. During the hinging process, the structure develops an increased load-carrying capacity above that corresponding to the formation of the initial plastic hinge. On the other hand, the members in which the early-forming hinges occur must be sufficiently stocky and well braced so that inelastic deformations can occur without loss of moment capacity.

Deflections at the specified load level are limited in accordance with Clause 6.2.1. This is based on the premise that where deflections are important the same limitations should apply regardless of the method of analysis. Plastically designed structures are usually "elastic" at specified load levels so the deflections would be computed on the basis of an elastic analysis.

8.5(a) Material. The plastic method relies on certain basic assumptions for its validity[12] and this Standard imposes the necessary restrictions in order to preserve the applicability of the plastic theory. The basic restriction pertains to the steel itself and is contained in Clause 8.5(a) which states, in effect, that the steel specified shall be characterized by a plateau in the stress-strain curve at the yield stress level and shall exhibit strain-hardening when the average strain exceeds the plastic strain. The use of steels exhibiting significant amounts of strain-hardening is a first step toward ensuring that satisfactory moment re-distribution will occur[13]. This behaviour should be evidenced at the temperatures to which the structure will be subjected in service. Also, although not explicitly stated, plastically designed structures usually entail welded fabrication, and therefore the steel specified should also be weldable. At normal temperatures all the steels referred to in Clause 5.1.2 should be satisfactory except for CSA G40.21-M, 700QT steels ($F_y > 0.80F_u$).

8.5(b) Width-Thickness Ratios. In order to preclude local buckling and thus ensure adequate hinge rotation, compression elements in regions of plastic moment must have width-thickness ratios no greater than those specified for Class 1 (plastic design) sections in Clause 11.3. Class 1 sections are more restricted in width-thickness ratio requirements than Class 2 sections. Although both are expected to meet the same strength requirement (attainment of the plastic moment), only plastic design sections need the rotation capacity necessary for redistribution of moments.

8.5(c) Lateral Bracing. The lateral bracing requirements are more severe than those for structures designed on the basis of an elastic moment distribution to ensure that adequate rotation capacity of the member is attained. Two values of L_{cr} are specified, one for the case where the moment gradient is pronounced, the other (more stringent) for the case of uniform, or near uniform moment. The dividing point has been selected as $M/M_p = 0.5$ as this best agrees with available test results[14]. Both criteria should be applied and the more severe requirement shall govern the determination of L_{cr}.

As the final hinge in the failure mechanism does not require rotation capacity the bracing spacing limitations of this clause do not apply in this case and the requirements of Clause 13.5.2 may be used.

Lateral bracing is required to prevent both lateral movement and twisting at a braced point. Lateral bracing is usually provided by floor beams or purlins which frame into the beam to be braced. These bracing members must have adequate axial strength and axial stiffness to resist the tendency to lateral deflection. These requirements are indicated in Clause 20.3 and further information on the design of bracing members is available in Reference 15. When the bracing member is attached to the braced member only at the compression flange, it is desirable that the lateral brace possess some bending stiffness about its major axis; however, there are insufficient experimental results to indicate the magnitude required.

A concrete slab into which the compression flange is embedded or to which the compression flange is mechanically connected, as in composite construction, or metal decks welded to the top flange of the beam in the positive moment region, would provide sufficient restraint to lateral and torsional displacements. Reference 107 suggests that the point of contraflexure of the plastic moment diagram may be considered a braced point. If the lateral brace is connected to the tension flange, provision must be made for maintaining the shape of the cross-section and for preventing lateral movement of the compression flange.

8.5(d) Web Crippling. Web stiffeners are required on a member at a point of load application where a plastic hinge would form. Stiffeners are also required at beam-to-column connections where the loads delivered by beam flanges would either cripple the column web or, in the case of tension loads, curl the column flange. The rules for stiffener design are given in Clause 21.3.

In lieu of pairs of stiffener plates parallel to and approximately in line with the flanges of the member delivering the load, plates parallel to the column web and attached to the toes of the column flanges may be used. Reference should be made to the technical literature[12] for further details of stiffeners and for special requirements pertaining to tapered and curved haunches[16].

When the shear force is excessive, additional stiffening may be required to limit shear deformations. The capacity of an unreinforced web to resist shear is assumed to be that related to an average shear yield stress equal to $F_y/\sqrt{3}$. The effective depth of the web of a rolled shape is taken as 95 per cent of the section depth. This leads to the expression, in Clause 13.4.2

$$V_r = 0.95\ \phi\ d\ w\ F_y/\sqrt{3} = 0.55\ \phi\ wd\ F_y$$

At beam-to-column connections, if the shear force exceeds that permitted by Clause 13.4.2, the deficiency may be overcome by providing doubler plates to increase the web thickness or by providing diagonal stiffeners (Figure 3). The force in the beam flange that is transferred into the web as a shear is

(approximately)

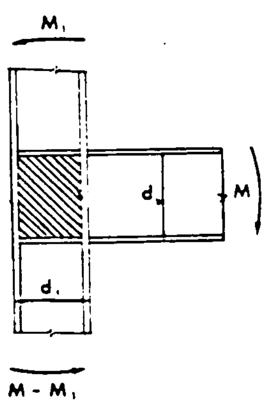
$$V = M/d_b$$

Equating this to the shear resistance as given in Clause 13.4.2 (where now $w = w_c$ and $d = d_c$), and solving for the required web thickness,

$$w_c > \frac{M}{0.55\phi\ d_c\ d_b\ F_y} > \frac{2.0\ M}{d_c\ d_b\ F_y}$$

Figure 3

WEB THICKNESS AT BEAM-TO-COLUMN CONNECTIONS



WITHIN HATCHED ZONE

$$w_c = \frac{2M}{d_c\, d_b\, F_y}$$

If $w < w_c$ provide diagonal stiffeners, doubler plate or other web reinforcement.

If the actual web thickness is less than w_c, the required area of diagonal stiffeners may be obtained by considering the equilibrium of forces at the point where the top flange of the beam frames to the column. The total force to be transmitted ($V = M/d_b$) is assumed to be taken by the web and the horizontal component of the force in the diagonal stiffener:

$$V = M/d_b = 0.55\ \phi\ w_c\ d_c\ F_y + \phi\ F_y\ A_s\ \cos\theta$$

where

A_s = cross sectional area of diagonal stiffeners, and

$\theta = \tan^{-1}(d_b/d_c)$

The required stiffener area is therefore

$$A_s = \frac{1}{\phi\ \cos\theta}\left[\frac{M}{F_y\ d_b} - 0.55\ w_c\ d_c\right]$$

8.5(e) Splices. The bending moment diagram corresponding to the failure mechanism reflects the changes in stiffness that occur during the plastic hinging process. For example, points of inflection in the final bending moment distribution may be required to resist significant moments to enable the structure to reach its predicted load-carrying capacity[17]. To ensure that splices have sufficient capacity to enable the structure to act as if continuous up to the ultimate load, a minimum connection requirement of 0.25 M_p is specified in Clause 8.5(e). At any splice location, the computed moments corresponding to various factored loading conditions must be increased by 10 percent; the splice is then designed either for these increased moments or for the minimum requirement of 0.25 M_p.

8.5(f) Impact and Fatigue. The use of moment redistribution to develop the strength of the structure corresponding to a failure mechanism implies a ductile type of behaviour. Members which may be repeatedly subjected to heavy impact and members which may be subject to fatigue should not be designed on the basis of a plastic analysis since ductile behaviour cannot be anticipated under these conditions. Such members, at least for the present, are best proportioned on the basis of an elastic bending moment distribution.

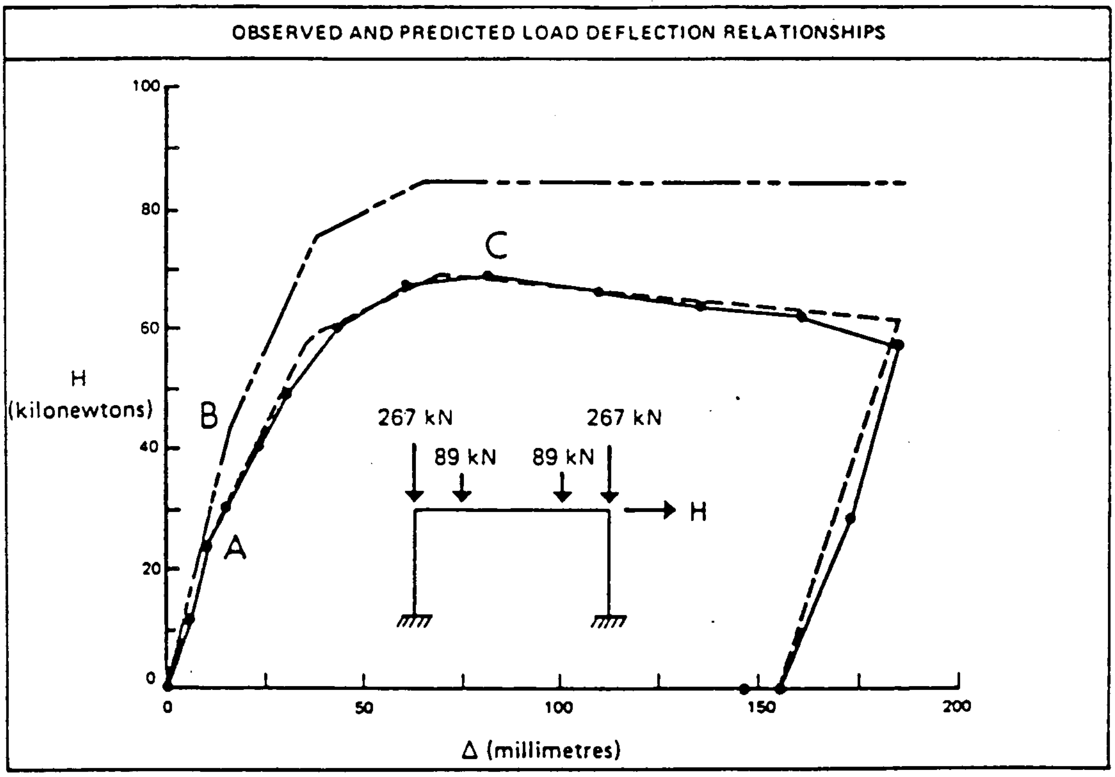
8.5(g) Inelastic Deformations. For continuous beams and certain types of relatively stiff, regular frames, the additional moments produced by the vertical loads acting through the lateral displacements of the structure may be negligible. For other types of structures, in particular multi-storey frames, these secondary effects may have a significant influence on the strength of the structure[12].

For example, in the structure shown in the inset of Figure 4, the secondary effects have reduced the ultimate strength for lateral loads by approximately 25 per cent[2,18]. In this structure the first plastic hinge formed at stage A while the ultimate strength (considering moment redistribution) was not attained until stage C. The inelastic deformations between these two stages have a serious influence on the strength of the structure. Clause 8.6.1 requires that the sway effects produced by the vertical loads be accounted for in design. The specific purpose of Clause 8.5(g) is to ensure that, in a structure analyzed on the basis of a plastic moment distribution, the additional effects produced by inelastic sway deformations are accommodated. In most cases the actual strength of the structure can only be predicted by tracing the complete load-deflection relationship for the structure or for selected portions[19]. Methods are available to perform this type of design. For braced multi-storey frames, however, simpler techniques have also been developed[20].

8.6 Stability Effects

8.6.1. The basic thrust of Clause 8.6.1 is the recognition that all structures are subjected to sway deformations. The vertical loads acting through the deformed shape of the structure produce secondary bending moments in the case of a moment resisting frame or additional forces in a vertical truss system. These additional moments or forces (the stability effects) reduce the strength of the structure, as shown for a moment-resistant frame in Figure 4. In addition, bending moments and deflections which

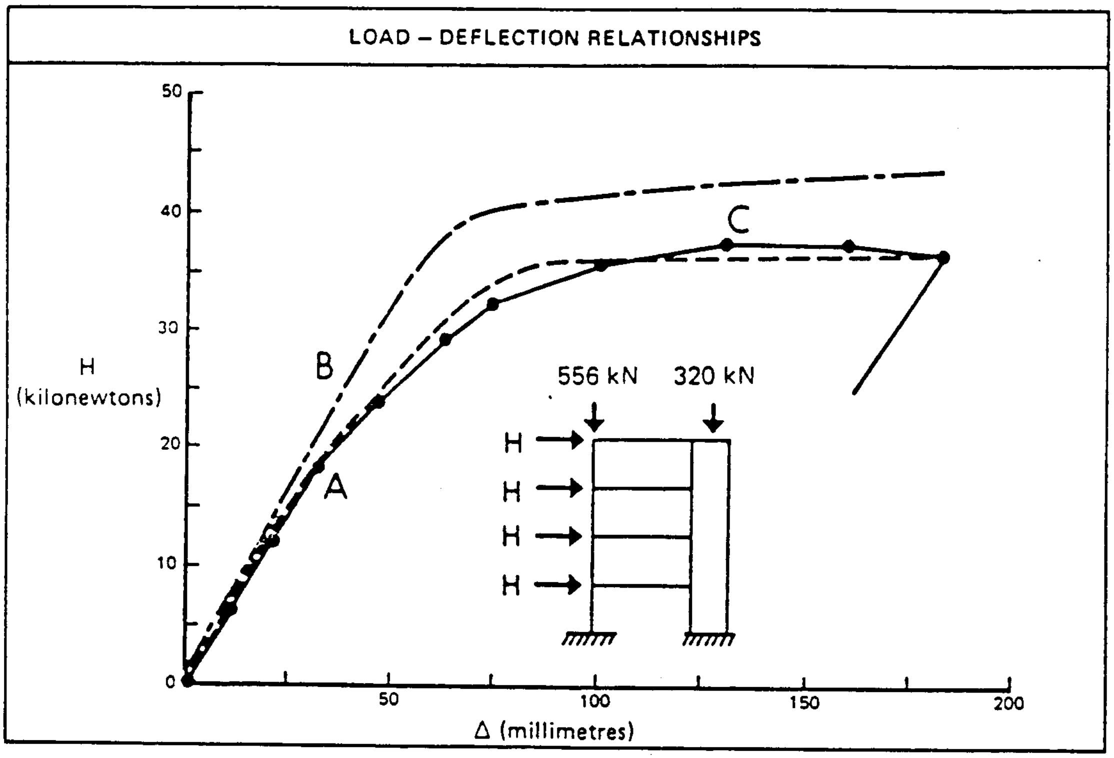
Figure 4



exceed those predicted by a first order analysis are produced at all stages of loading[18]. Similar effects are produced in structures containing a vertical bracing system, as shown in Figure 5 where the steel frame is linked to a reinforced concrete shear wall[18].

To account for these stability effects, two different approaches are possible. The approach recommended is to perform analyses which include the stability effects. This type of analysis is termed a second-order analysis, since equilibrium is formulated on the deformed structure. In this way the additional moments or forces generated by the vertical loads are

Figure 5



accommodated directly. An alternative approach is provided through the use of Clause 8.6.3.

In some cases these secondary effects are small and may be ignored by the designer. Presumably this will be the case for relatively stiff structures subjected to relatively small vertical loads and having a reasonably uniform distribution of lateral load resisting elements. Although studies are in progress to determine the type of structures that would fall into this category the results are not yet available. The onus is clearly on the designer to assure himself that the secondary effects are negligible if they are not to be included in the design process.

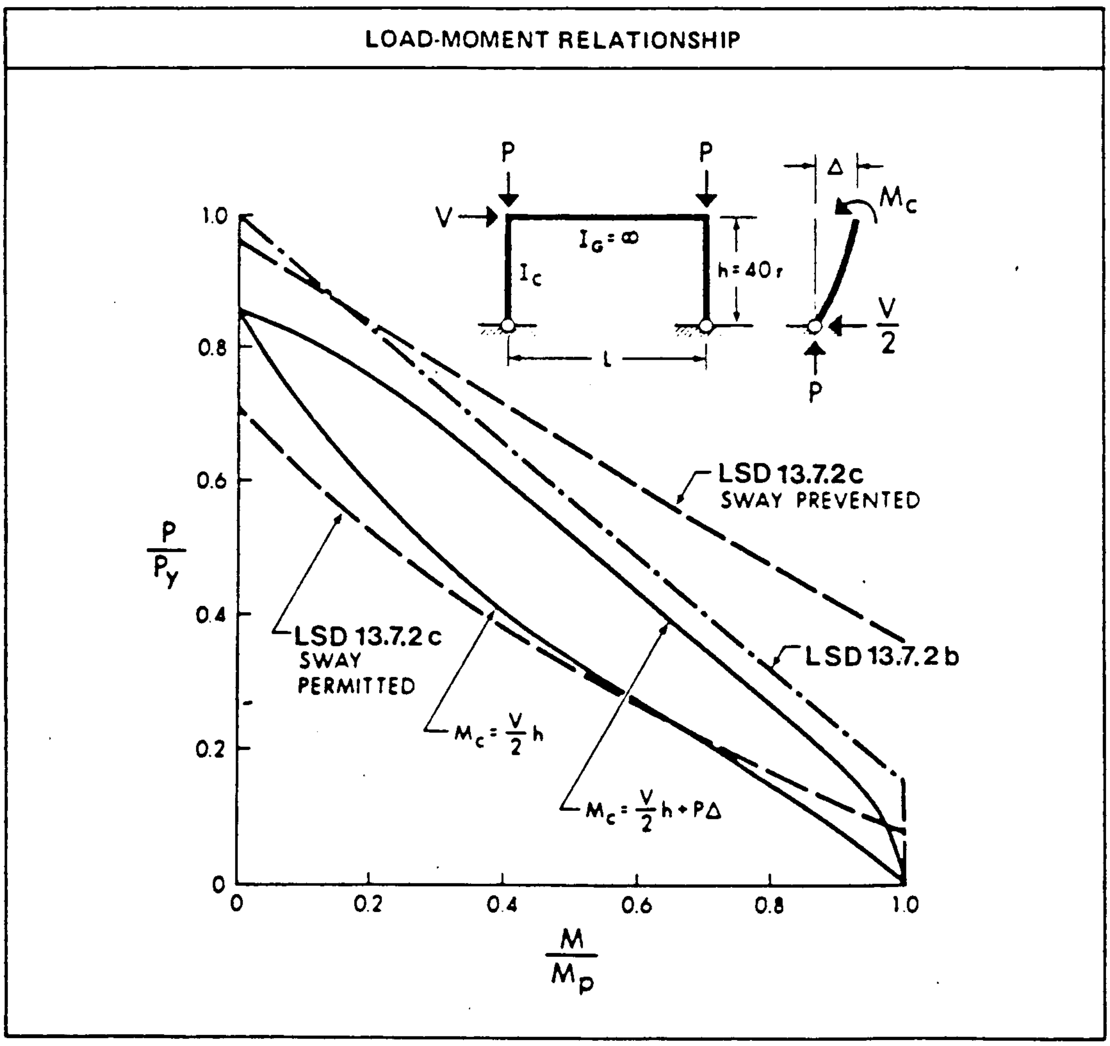
8.6.2 To determine the magnitude of the stability effects two approaches are suggested. Computer programs are available, based on equilibrium of the deformed structure[21,22]. The use of this type of program ensures that the additional moments or forces generated by the vertical loads acting through the displaced structural shape (the so-called $P\Delta$ effect) will be taken into account. In addition, most second-order programs also account for the reduction in column stiffnesses, caused by their axial loads[22].

The second approach, outlined in Appendix I is simply to modify the results of first-order analysis to include the $P\Delta$ effects. It is implied in this approach that the reduction in individual member stiffnesses will be negligible, although a simple check may be used to ensure that this assumption is justified[18].

In Appendix I a technique is outlined whereby the deflections computed by a first-order analysis may be used to compute artificial sway forces[18]. These forces are then added to the original forcing system and the structure is reanalyzed. The final moments or forces then include an allowance for the $P\Delta$ effects.

Appendix I calls for an iterative process to be used to ensure that the $P\Delta$ effects are not underestimated. As an alternative, estimated deflections may be used to calculate initial sway forces[92]. The structure is then analyzed under the lateral load caused by wind or earthquake plus the sway forces. If the resulting deflections are less than those assumed for the initial estimate of the PΔ effects then these effects have been over-estimated and (if the designer is satisfied that the situation is acceptable) the iterative process is not required.

Figure 6



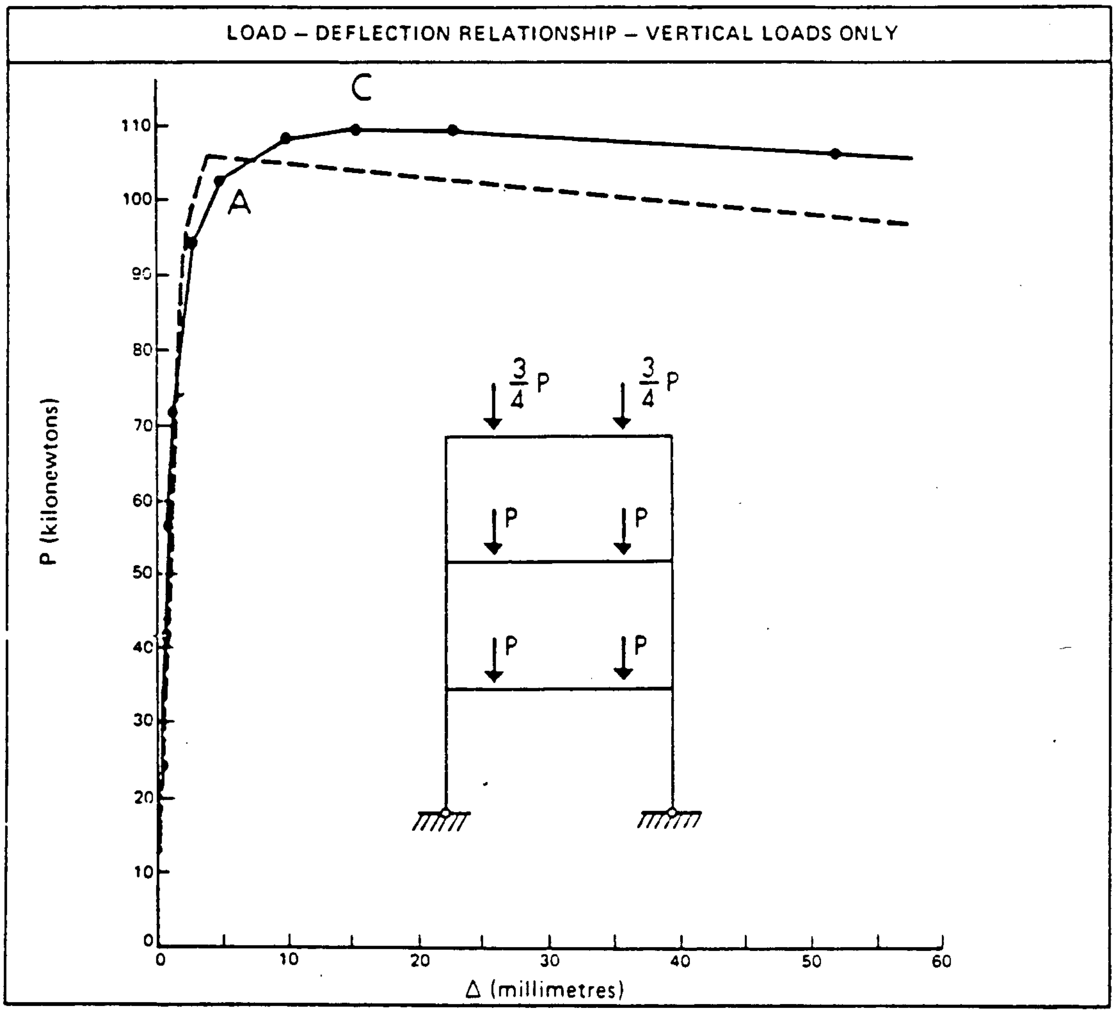
In this direct procedure a check on the flexibility of the structure (as required by step 6 of Appendix I) is not achieved. It has been observed[25], however, that most regular structures meeting reasonable deflection limits under the specified loads will be adequately designed using the $P\Delta$ approach.

Those structures for which the $P\Delta$ effects have been included in the analysis, or for which these effects are negligible, may be designed on the basis of the sway-prevented condition. The basis for this is shown in Figure 6 where the ultimate strengths are plotted for the frame shown in the inset. The results plotted as the solid curves relate the maximum value of the column top moment, M_c, to the axial load ratio, P/P_y. The primary bending moment ($M_c = \frac{Vh}{2}$) is shown as the lower solid curve while the total bending moment ($M_c = \frac{Vh}{2} + P\Delta$) is shown as the upper solid curve. The difference in moments between the two curves is then a measure of the $P\Delta$ effect.

The LSD stability interaction equation, 13.7.2(c), based on sway-prevented conditions and the LSD strength interaction, 13.7.2(b), are again plotted as the upper dashed line and the broken line in Figure 6. The envelope provided by these two relationships predicts the full moment capacity of the member. In this case, however, the prediction is slightly unconservative.

Clause 8.6.2 requires that the calculated $P\Delta$ effects be based on the deflections produced by the applied loads (Clause 8.6.2(a)) magnified by the flexibility of the frame. In some cases, particularly for combinations of vertical loads only, significant lateral deflections will not be developed.

Figure 7



For these cases, however, the PΔ effects are still important, as shown in Figure 7. The frame, shown in the inset to the figure, is subjected to vertical load only. As the loads are increased, the PΔ effects produced by the vertical loads acting through the lateral displacements caused by fabrication lead to failure through instability, much the same as the combined loading cases shown in Figures 4 and 5.

To simulate this condition, Clause 8.6.2(b) requires that the PΔ effects be based on the initial out-of-plumbness produced during the erection process. The corresponding tolerances are specified in Clause 29.7.1 and References 18, 23 and 92 illustrate the calculation of the PΔ effects.

Note that the more severe of the requirements of Clause 8.6.2(a) or 8.6.2(b) will govern the design. It is not considered necessary to add the two effects together[18].

Clause 8.6.2(c) emphasizes the fact that out-of-plumbness which are different in sense in adjacent storeys may lead to the more severe requirements for the design of beam-to-column connections, etc.

In dealing with the forces produced by out-of-plumbness, the conservative position was taken, namely that all columns in a given story will be deflected into this position causing the most severe effect. Information on actual, statistical out-of-plumbness is available[109].

8.6.3. As an alternative to the above approach, the analysis used by the designer may be a first-order analysis, in other words the moments produced by the PΔ effect are not computed. The stability interaction equation, LSD Equation 13.7.2(c), based on sway-permitted conditions, is plotted as the lower dashed line in Figure 6. It is apparent that the stability interaction equation used in this fashion does not predict the full capacity of the member. Rather, the equation limits the capacity available to the designer to only that portion used by the primary bending moment. The remaining member capacity is then available to resist the PΔ effect (which is presumably not computed nor included in the analysis).

In 2.7B - Working Stress Design, it is assumed that if the structure contains a stiff vertical truss or shear wall translation is effectively prevented and the columns can be designed as sway-prevented members. In the absence of this stiff vertical element, the columns are to be designed as if sway is permitted. The real distinction between the sway-permitted case and the sway-prevented case, however, is that in the latter the interaction equations permit the designer to utilize the full moment capacity of the column while, if the column is designed as sway-permitted, the designer is able to use only a part of the full capacity.

2.7A - Limit States Design requires the use of the interaction equations based on sway-permitted conditions if the $P\Delta$ effects have not been included in the analysis. It should be noted that this approach is not recommended for structures having columns of significantly differing stiffnesses and is not permitted for structures resisting lateral loads by vertical truss or shear wall systems[24].

9. DESIGN LENGTH OF MEMBERS

9.1 For design purposes it is usually convenient to consider the length of a member as equal to the distance between centres of gravity of supporting members. In most instances the difference resulting from considering a member to be that length rather than its actual length centre-to-centre of end connections is small. In some cases, however, there is sufficient difference to merit computing the actual length. Regardless of the length used for design, the actual connection detail may cause an eccentric load, or moment, to act on the supporting member and this effect must be taken into account.

9.3 Compression members are designed on the basis of an effective length (KL) which is the product of the actual unbraced length and the appropriate effective length factor. Both the unbraced length and the effective length factor may vary with respect to the cross-sectional axis of the member under consideration.

The concept of effective length (KL) is used in computing the slenderness ratio of compression members, and hence, in determining the resistance of compression members. Much information about effective lengths is contained in the technical literature[25,26] and some guidance for the designer is provided in Appendices B and C of the Standard. The CISC-CSCC Column Selection Program 3 (a computer program available to designers)[27] contains routines for computing effective lengths, based on the principles outlined in Appendix C.

The major difference between 2.7B - Working Stress Design and the requirement of Clause 9.3 of Part 1 - Limit States Design is the distinction made between the conditions for which the effective length factor should be computed on the basis of a sway-prevented model and those for which the sway-permitted model is appropriate. The commentary on Clause 8.6 provides guidance on this question.

10. SLENDERNESS RATIOS

The maximum slenderness ratio of 200 for compression members, stipulated in Clause 10.2.1, has been included for practical reasons. As illustrated in Appendix D the strength, or resistance, of a compression member becomes quite small as the slenderness ratio increases beyond about 150, and the member becomes relatively inefficient. Accordingly, a limiting permissible slenderness ratio of 200 was established.

In theory, from considerations of strength, no limiting slenderness ratio is required for a tension member. Again, however, considerations of serviceability resulted in the imposition of a slenderness ratio limit of 300, with permission to waive this limit under specified conditions.

11. WIDTH-THICKNESS RATIOS: COMPRESSION ELEMENTS

Clause 11.1 identifies four categories of cross-sections, Class 1 through Class 4, based upon the width-to-thickness ratios of the elements of the cross-sections in compression. The ratios given in Table 1 of Clause 11 for Classes 1, 2, or 3 ensure that the respective ultimate limit states will be attained prior to local buckling of the plate elements. These ultimate limit states are: Class 1 - attainment of the plastic moment capacity (beams) or the plastic moment capacity reduced for the presence of axial load (beam-columns) along with rotation capacity sufficient to fulfill the assumption of plastic analysis;
Class 2 - attainment of the plastic moment capacity as above for

beams and beam-columns but with no requirement for rotation capacity; Class 3 - attainment of the yield moment for beams or the yield moment reduced for the presence of axial load for beam-columns.

For axially loaded members such as columns or struts, the distinction based on moment capacity does not exist. Table 1 simply shows the same limit for each of Classes 1, 2, and 3 for webs in axial compression. The flange limit for columns should be taken as that for Class 3 beam flanges, that is, $200/\sqrt{F_y}$.

Figure 8 illustrates the requirements for Classes 2 and 3 which sections will be those most used in practice.

The basis of the requirements, particularly those for W-shapes, come from a background of both experimental and theoretical studies. For example, the restrictions on flanges have both a theoretical basis (see, for instance, Ch. 6 of Ref. 29 or Art. 6.2 of Ref. 12 or Ch. 4 of Ref. 25) and an extensive experimental background[30,31,32]. The restrictions for webs in flexural compression come from both theory and tests for Class 1[30] but mostly from test results for Classes 2 and 3[33,34].

Figure 9 shows the requirements for the case of webs under both axial compression and compression due to bending. Since the amount of web under compression varies from complete (columns) to one-half (beams), the depth-to-thickness limits will vary as a function of the amount of axial load. The basis for Class 1 beam-columns is derived from early work on plastic design[30] while the limits for Classes 2 and 3 come mostly from experimental studies[35,36].

Figure 8

WIDTH – THICKNESS RATIOS FOR COMPRESSION ELEMENTS

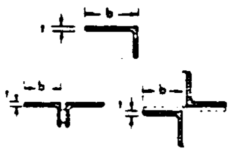
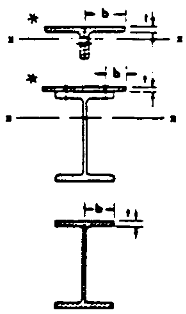
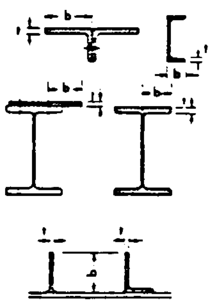
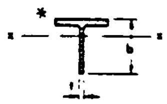
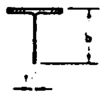
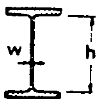
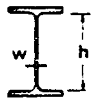
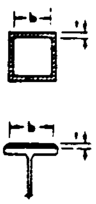
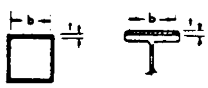
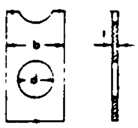
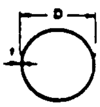
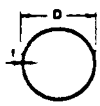
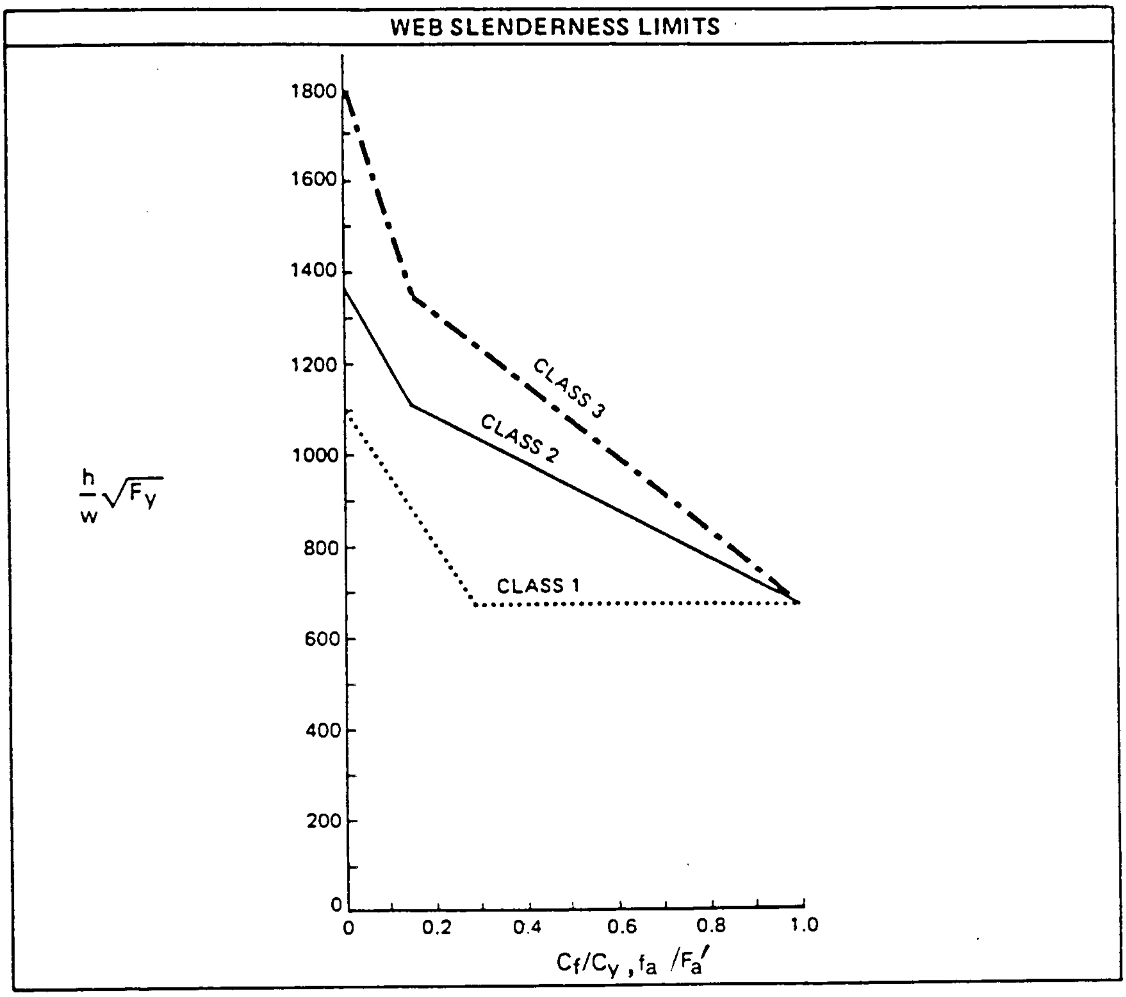
CLASS 2 SECTIONS		CLASS 3 SECTIONS	
NOTE: Class 2 sections must have an axis of symmetry in plane of bending. Sections marked * are considered Class 2 only for bending about x-x.			$\frac{b}{t} \leqslant \frac{200}{\sqrt{F_y}}$ 'b' is the longer leg length
	$\frac{b}{t} \leqslant \frac{170}{\sqrt{F_y}}$		$\frac{b}{t} \leqslant \frac{200}{\sqrt{F_y}}$
	$\frac{b}{t} \leqslant \frac{170}{\sqrt{F_y}}$		$\frac{b}{t} \leqslant \frac{340}{\sqrt{F_y}}$
	$\frac{h}{w} \leqslant \frac{1\,370}{\sqrt{F_y}}$ bending only $\frac{h}{w} \leqslant \frac{670}{\sqrt{F_y}}$ axial comp^n.		$\frac{h}{w} \leqslant \frac{1\,810}{\sqrt{F_y}}$ bending only $\frac{h}{w} \leqslant \frac{670}{\sqrt{F_y}}$ axial comp^n.
	$\frac{b}{t} \leqslant \frac{525}{\sqrt{F_y}}$		$\frac{b}{t} \leqslant \frac{670}{\sqrt{F_y}}$
			$\frac{b}{t} \leqslant \frac{840}{\sqrt{F_y}}$ Effective Area = (b - d) t
	$\frac{D}{t} \leqslant \frac{18\,000}{F_y}$		$\frac{D}{t} \leqslant \frac{23\,000}{F_y}$

Figure 9



Recent studies at the University of Alberta attempting to unify this work in respect of W-shapes indicate that a slight upward revision or no change in most categories is in order.[140]

Sections used for columns, beams, or beam-columns may be composed of elements whose width-to-thickness ratios exceed those prescribed for Class 3 provided that the resistance equations are adjusted accordingly. These sections, called Class 4, should be evaluated according to the rules given in Clause 13.

The requirement for Class 3 circular hollow sections, $D/t \leq 23{,}000/F_y$ is based on tests[37] which indicates that tubes that satisfy this requirement can reach the yield stress without local buckling. The more conservative requirements for Class 1 and Class 2 sections are based on research referred to in Reference 28.

12. GROSS AND NET AREAS

12.3 Net Area

When bolt holes are punched the net area shall be determined on the assumption that the hole diameter is 2 mm greater than that specified by the designer to provide an allowance for the distortion that occurs in the metal around a punched hole. As holes for fasteners are customarily required to be 2 mm greater than the fastener diameter, Clause 23.3.2, the net area is determined on the basis that punched holes are 4 mm greater in diameter than the fasteners that will be used.

The location of the least width of a part may be obvious from inspection. When it is not clear which of several potential tear paths might be critical, the various possibilities must be examined, and the net width established using the "$s^2/4g$" rule (Clause 12.3.3).

Both tests and theory show that once any hole or pattern of holes has been introduced into a part, there is an upper limit to the amount of material that will be effective in resisting tensile loads. This limit is principally governed by the ratio of the yield point of the steel to its ultimate strength. For the types of steels most commonly used at present in structural work (e.g., G40.21-M 300W), the maximum net area that can be considered effective is limited to 85% of the gross cross-sectional area (Clause 12.3.4a). Other limits, higher than this value, are specified for higher strength steels.

12.4 Pin-Connected Tension Members

In pin-connected tension members the non-uniform distribution of stress makes it desirable that the net area across the pin hole be at least one third greater than the area of the body of the member[38]. To avoid end splitting, the area beyond the pin hole within a 45 degree arc each side of the longitudinal axis of the member must be at least 90 percent of the area of the body of the member.

13. MEMBER AND CONNECTION RESISTANCE FOR LIMIT STATES DESIGN OR ALLOWABLE STRESSES FOR WORKING STRESS DESIGN

13.2 Axial Tension

The resistance (LSD) or allowable stress (WSD) of a tensile member is to be established as the lesser of the capacity based on yield strength (Clause 13.2(a)(i)) and the capacity based on ultimate strength (Clause 13.2(a)(ii)).

The yield load capacity is further distinguished as to the amount of ductility that might be expected to occur in members that use mechanical fasteners rather than welds. In such cases, if the ratio of net area to gross area (A_n/A_g) is greater than the ratio of the yield point of the material to its ultimate strength (F_y/F_u), then the member cross-section will yield on the gross cross-section prior to the time that the ultimate strength is reached at the net cross-section (that is, $A_nF_u>A_gF_y$). Such a member would have considerable reserve ductility if failure were to occur. Load could possibly be redistributed to other parts of the structure but, in any event, failure would be preceded by ample warning as the result of large deformations.

If the criterion stated above is not met, failure would potentially occur by tearing through the net cross-section before any significant amount of yielding was present (that is, $A_nF_u<A_gF_y$). When fracture might precede yielding, a lower tensile resistance (LSD) or allowable tensile stress (WSD) is specified. It varies with the amount of ductility that might be expected, that is, with the ratio A_n/A_g.

Clause 13.2(a)(ii) places a limit on the tensile capacity based on ultimate strength. The limit here is somewhat more stringent than that placed on yield strength because failure at ultimate is absolute. The multiplier 0.85 in the LSD equation is introduced to take this into account and parallels WSD where the nominal factor of safety against failure by yielding is 1.67, and against ultimate tensile capacity is 2.0 (0.85 is approximately equal to the ratio of 1.67 to 2.00).

The resistance in LSD provided by a pin-connected member is to be taken as 0.75 times that for a member connected by the usual structural fastener. In WSD the allowable axial tensile stress on the net area across the pin holes is limited to $0.45F_y$, that is, 0.75 times the basic allowable tensile stress of $0.60F_y$. This reduction recognizes the greater non-uniformity of stress that occurs around a hole that is relatively large.

The allowable axial tensile stress (WSD) on the unthreaded body area of threaded parts is limited to $0.40F_y$. This assures that the stress on the net section through the threads does not exceed $0.60F_y$. If bars, etc., have upset threads, F_t is not to exceed $0.60F_y$ on the critical net area which may be through the threads or in the body.

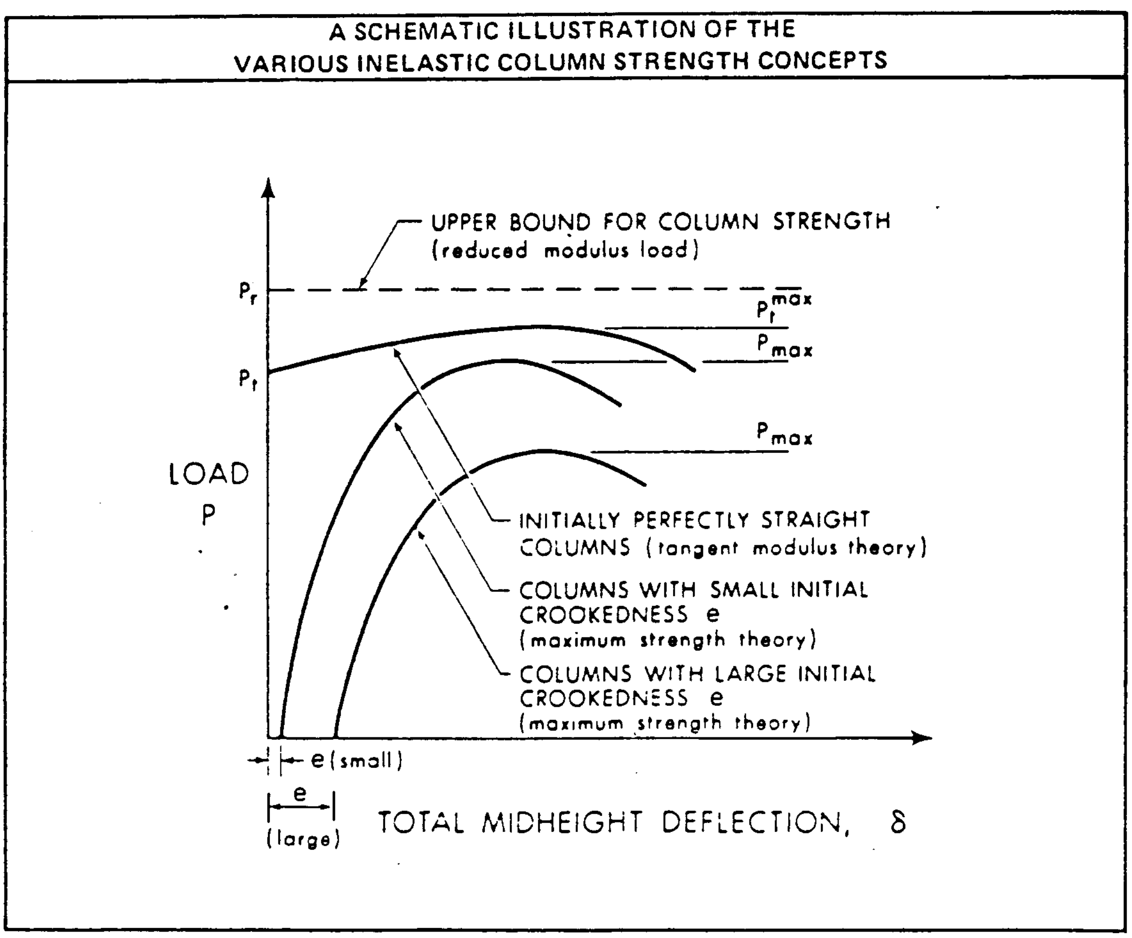
13.3 Axial Compression

Steel columns are conveniently classified as short, intermediate, or long members, and each category has an associated characteristic type of behaviour. Loosely speaking, a short column is one which can resist a load equal to the yield load ($C_y = AF_y$) or can sustain stresses at least equal to the yield stress level. Thus the strength is governed only by the yield stress of the steel. The failure of long columns is accompanied by a rapid increase in lateral deflection and the member is slender enough so that the load at which this rapid deflection takes place does not produce significant yielding of the cross-section. The maximum load is not a function of the material strength but depends on the bending stiffness (EI) and length of the member.

Columns in the intermediate range are the most common group in steel buildings and are the most difficult to analyze. Failure is again characterized by a rapid increase in deflection but this takes place at a load when some portions of the cross-section have yielded. The amount of yielding that takes place is greatly influenced by the residual stresses that are present and the strength of the column is influenced both by this and by the magnitude of the initial imperfections.

Both the effects of residual stress and initial out-of-straightness are considered in formulating the relationship for the column strength. In the place of the relatively simple tangent modulus theory used to predict the ultimate strength of the perfectly straight column[29], the member must be treated as a beam-column acted upon by an axial force and by bending moments that vary along the length of the member[40].

Figure 10



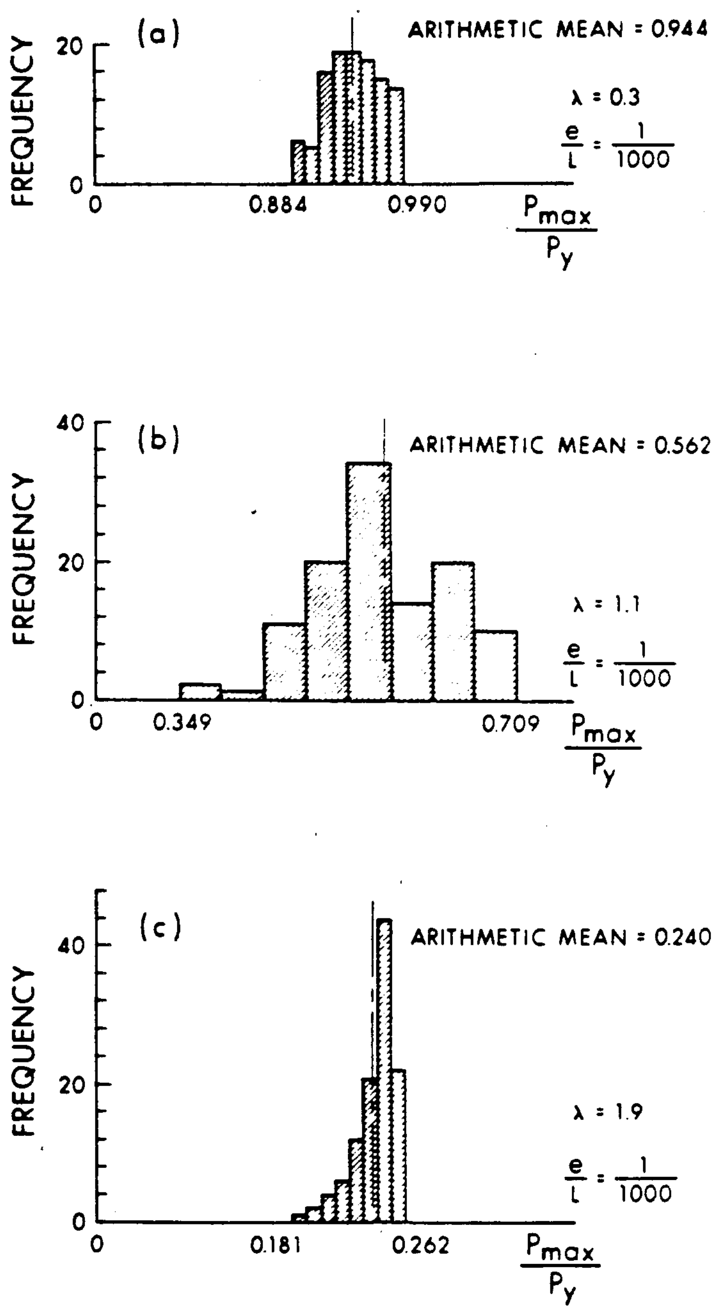
The behaviour of a column predicted by the two different approaches is shown qualitatively in Figure 10 where the load, P, is plotted against the mid-height lateral deflection δ. For columns having relatively small initial out-of-straightness, the maximum strengths may be approximately the same as the tangent modulus buckling loads. As the initial out-of-straightness increases however, the maximum strength of the member drops significantly below the tangent modulus buckling load.

This strength reduction may be particularly severe for columns in the "intermediate" range of slenderness ratios. For those members, inelastic action, where some of the stresses due to the axial load plus bending moment added to the compressive residual stresses reach the yield stress, occurs well before the maximum strength is attained. Thus, the magnitude of the residual stresses as well as the magnitude and shape of the out-of-straightness significantly influence the results. Figure 11 gives some indication of the scatter obtained from analyses of columns having the same out-of-straightness but differing residual stress distributions.

In order to reflect the various factors affecting the maximum strength of columns having various slenderness ratios, column curves proposed by the Structural Stability Research Council[25], have been adopted. The Council's Column Curve 2 is used as the basis for the description of resistance of W-shapes and for cold-formed non-stress relieved hollow structural sections (Clause 13.3.1)[114].

Figure 11

TYPICAL FREQUENCY DISTRIBUTION HISTOGRAMS FOR THE MAXIMUM STRENGTH OF ALL 112 COLUMN CURVES WITH INITIAL OUT-OF-STRAIGHTNESS e/L = 1/1000



The footnote to Clause 13.3.1 should be reviewed carefully by designers. The column curve given should be used only for W-shapes up to 610 mm deep or for other doubly-symmetric Class 1, 2, or 3 sections except that solid round non-stress relieved cold straightened bars are specifically excluded. The curve should be used for welded H-shapes only when the flange edges have been flame cut. (The treatment of other welded H-shapes is discussed below.) Because the expressions for column resistance (LSD) or allowable compressive stress (WSD) are for x-axis or y-axis buckling only, singly symmetric, asymmetric, or cruciform sections should also be checked for lateral-torsional buckling.

Because of a more favorable residual stress pattern, the resistance (LSD) or allowable stress (WSD) of hot formed or cold formed stress relieved hollow structural shapes (Class H) is determined from Column Curve 1 of the Structural Stability Research Council (Clause 13.3.2)[112].

Many H-shaped columns, usually called WWF shapes, are made by welding together three plate components. The flange to web welds induce large residual stresses which are tensile in the vicinity of the weld and compressive near the flange tips. Unless offsetting tensile residual stresses exist in the plate which comprises the flange tips, before the section is welded, the compressive stresses can be higher than normally occur in rolled shapes, resulting in a section with reduced inelastic buckling strength. Studies show that, welded H-shapes to be used as columns, should be designed in accordance with the formulas in Clause 13.3.1 only if the flange edges are flame cut. The WWF sections produced in Canada are made from flame cut plate.

For many heavy sections and welded sections fabricated from universal mill plate, the Council's Column Curve 3 could be used, permitting a capacity reduced below that corresponding to Column Curve 2[25].

Because the column curves are based, in part, on the magnitude and distribution of residual stresses care should be exercised in their application, for example, in determining the capacity of an existing column which is being reinforced in such a manner that there is an increase in compressive residual stresses in the fibres most remote from the centroid. In such a situation a common solution would be to add material so as to greatly reduce the slenderness ratio.

13.4 Shear

13.4.1 Elastic Analysis

Although bending can be present unaccompanied by shear, a transversely loaded beam will always have shear combined with moment. However, in regions where shear predominates, it can be considered to act alone and Clause 13.4.1 sets out the appropriate equations. (The interaction of shear and moment is treated in Clause 13.4.6) The shear strength equations are set out for a stiffened plate girder: unstiffened plate girders or stocky rolled beams become special cases of this.

Extensive theoretical and experimental studies of plate girders[42] have indicated the following:

1. At stresses below the proportional limit (taken as 80% of the shear yield stress), buckling is elastic (Euler buckling).

2. At stresses above the proportional limit, buckling is inelastic.

3. A transition is required in the strength descriptions between the region where elastic buckling governs and the region where inelastic buckling controls. This is necessary principally because of the presence of residual stresses.

4. Considerable post-buckling strength is available in stiffened webs through the development of a tension field along the diagonal of a buckled panel.

5. Additional strength due to strain-hardening is attained in stocky webs.

The four ranges of resistance prescribed in Clause 13.4.1 correspond to the following modes of behaviour and are illustrated in Figure 12 for LSD and 13 for WSD.

1. Strain-hardening.

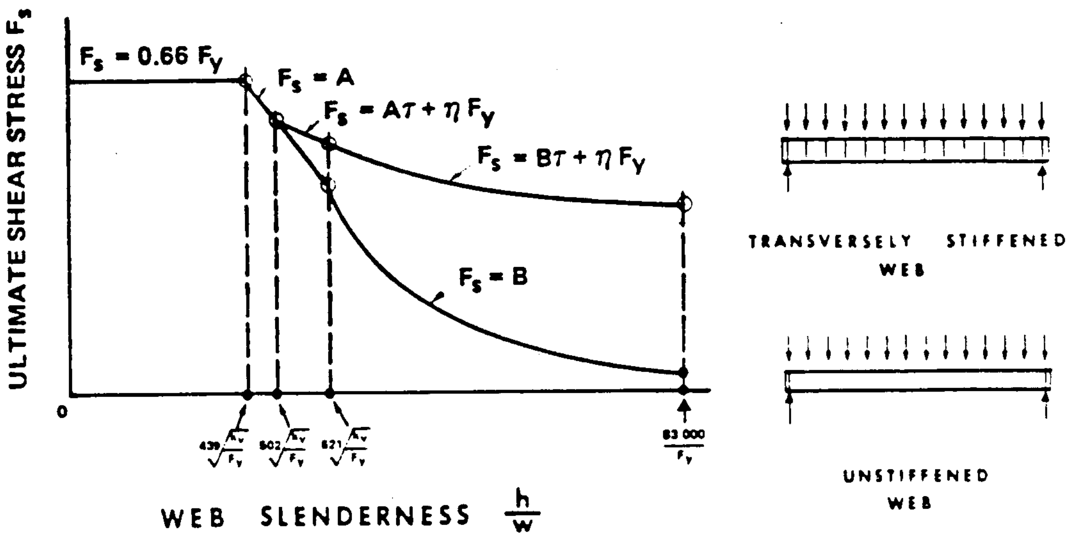
$$\frac{h}{w} \leq 439\sqrt{\frac{k_v}{F_y}}$$

2. Transition curve between strain hardening and inelastic buckling. At

$$\frac{h}{w} = 502\sqrt{\frac{k_v}{F_y}},$$

Figure 12

ULTIMATE SHEAR STRESS — WEBS OF FLEXURAL MEMBERS



$$A = \frac{290\sqrt{k_v\ F_y}}{h/w}$$

$$k_v = 4 + \frac{5.34}{(a/h)^2} \text{ when } \frac{a}{h} < 1$$

$$\tau = 1 - \frac{0.866}{\sqrt{1 + (a/h)^2}}$$

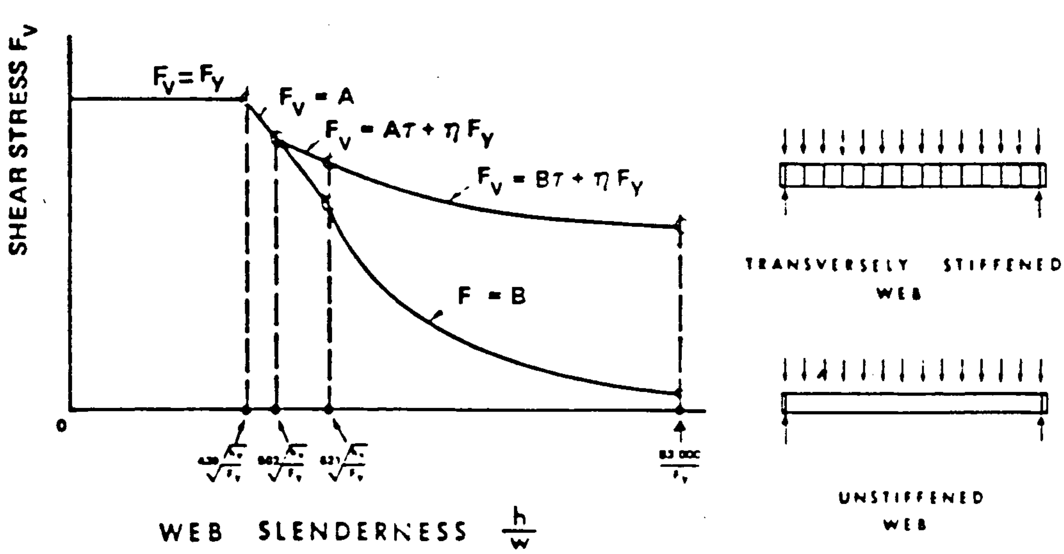
$$k_v = 5.34 + \frac{4}{(a/h)^2} \text{ when } \frac{a}{h} \geqslant 1$$

$$\eta = \frac{0.50}{\sqrt{1 + (a/h)^2}}$$

$$B = \frac{180\ 000\ k_v}{(h/w)^2}$$

Figure 13

ULTIMATE SHEAR STRESS – WEBS OF FLEXURAL MEMBERS



$$A = \frac{175\sqrt{k_v F_y}}{h/w}$$

$$k_v = 4 + \frac{5.34}{(a/h)^2} \text{ when } \frac{a}{h} < 1$$

$$\tau = 1 - \frac{0.866}{\sqrt{1 + (a/h)^2}}$$

$$k_v = 5.34 + \frac{4}{(a/h)^2} \text{ when } \frac{a}{h} \geq 1$$

$$\eta = \frac{0.50}{\sqrt{1 + (a/h)^2}}$$

$$B = \frac{109000\, k_v}{(h/w)^2}$$

$F_s = 0.577\ F_y$, which is the shear yield stress according to the von Mises-Hencky yield criterion.

$$439\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 502\sqrt{\frac{k_v}{F_y}}$$

3. Inelastic buckling, with post-buckling strength in stiffened webs.

$$502\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 621\sqrt{\frac{k_v}{F_y}}$$

4. Elastic buckling, with post-buckling strength in stiffened webs.

$$621\sqrt{\frac{k_v}{F_y}} < \frac{h}{w}$$

The upper limit in LSD on the shear stress of 0.66 F_y for the load carrying capacity of webs with low slenderness ratios is established by taking into account the beneficial effect of strain-hardening in the web. This limit corresponds to excessive deformation rather than to catastrophic failure and is higher than that derived from rational analysis, 0.55 F_y, as used in Clause 13.4.2 for plastic analysis. The value of $F_s = 0.66\ F_y$ corresponds to the allowable stress of 0.40 F_y used in Part 2 - Working Stress Design.

In unstiffened webs, no tension field can develop, and the second and third formulae become identical. In that case, the stiffener spacing "a" is taken as infinity and the significant parameters take the following values: $\tau = 1$, $\eta = 0$, $k_v = 5.34$.

In computing the shear resistance (LSD) or allowable shear stress (WSD) it is assumed that the shear stress is distributed uniformly over the depth of the web. The web area A_w is the product of web thickness (w) and web depth (h) except for rolled shapes where it is customary to use the overall beam depth (d) in place of (h).

13.4.2 Plastic Analysis

The shear resistance of unreinforced webs of flexural members designed on the basis of a plastic analysis is derived from a rational analysis assuming that the shear capacity is attained when the web is stressed uniformly at the level of $F_y/\sqrt{3} = 0.577F_y$ (von Mises-Hencky yield criterion).

Only 95% of the depth of the web is assumed to be effective, but since it is convenient to work with the nominal web area (dw), this factor is introduced directly into the numerical coefficient.

As pointed out in the preceding section, this value is lower than that used for elastic analysis; this is related to the strong probability that high shear and high moment will occur simultaneously at a hinge location, competing for the yield resistance of the web. There is enough interaction between the two to warrant retaining this limit until new evidence indicates that it may safely be increased.

13.4.3 Maximum Slenderness

To prevent the web from buckling under the action of the small vertical components of the flange force which arise as the girder is bent in flexure, the web slenderness, h/w, is limited to $\frac{83,000}{F_y}$. This limit is derived from the theoretical limit equal to $\frac{0.48\ E}{\sqrt{F_y(F_y + F_{rc})}}$, assuming F_{rc}, the residual stress in compression flange, to be approximately 1/3 F_y[29].

13.4.6 Combined Shear and Moment in Girders

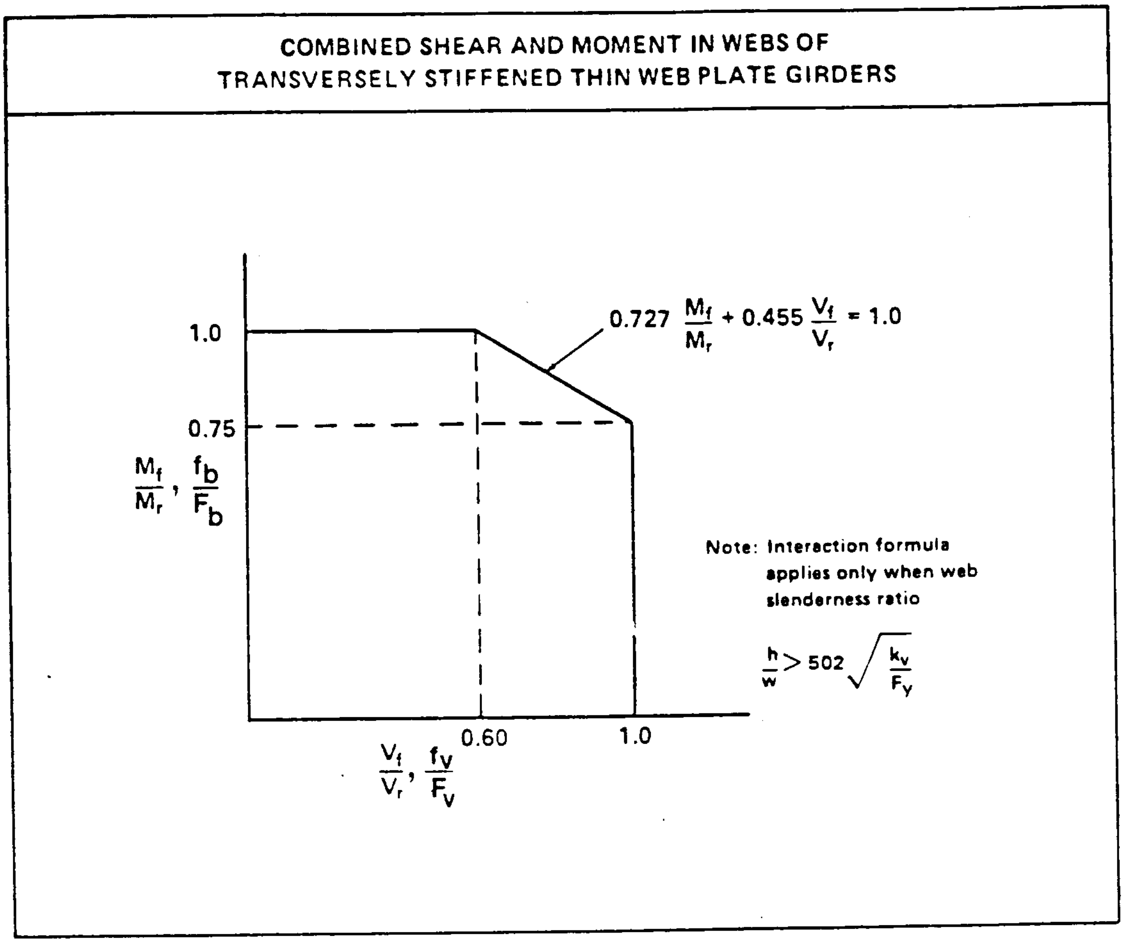
When high shear and high moment occur simultaneously in the web of a transversely stiffened girder, the available resistance to shear and moment (LSD) or the allowable combined stresses due to shear and moment (WSD) must be reduced according to the formula given in Clause 13.4.6. The intent is to prevent exceeding the yielding capacity of the web in the vicinity of the flange[44].

Girders are assumed to depend on tension field development only if the web slenderness ratio h/w > $502\sqrt{\frac{k_v}{F_y}}$ as defined in Clause 13.4.1. Figure 14 illustrates the interaction formula provided in Clause 13.4.6.

13.5 Bending - Laterally Supported Members

The moment resistance (LSD) or the allowable bending stress of a laterally supported member subjected to bending is dependent on the width-to-thickness ratios of the plates composing the cross-sections. Excessively slender plates will decrease the

Figure 14

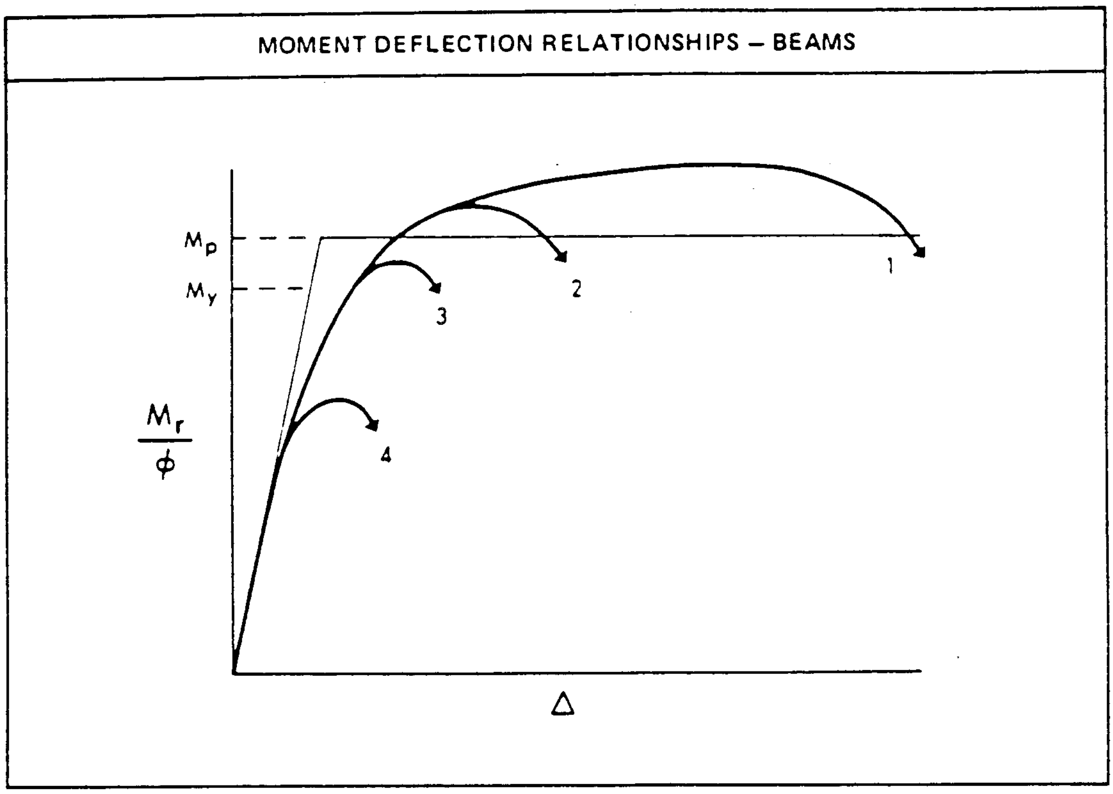


resistance of the plate element to buckling and will therefore reduce the moment capacity below the desired capacity, M_p, or M_y.

In a beam, local buckling of the compression flange is of primary concern although in built-up plate girders web buckling is also of importance.

In Figure 15 the moment resistance (determined by LSD equations and divided by ϕ) is plotted against a characteristic deflection for beams having various plate width-to-thickness

Figure 15



ratios. The curve shown as characteristic of Class 1 sections represents ideal behaviour for a beam. The moment resistance (LSD) reaches M_p and increases slightly as the beam continues to deflect. Eventually after a significant amount of inelastic deflection has occurred, local buckling of either the flange or web may occur, leading to a drop-off in moment capacity. The limitations on width-to-thickness ratios for Class 1 sections are given in LSD and WSD Clauses 11.3 for those sections meeting the requirements of Clause 11.1.3. The moment resistance (LSD) or the allowable bending stress (WSD) is based on the specified modulus. For rounds and rectangles, therefore allowable stresses of $0.90F_y$ and $0.75F_y$ are given.

Sections having plate components that are too slender to meet the requirements for Class 3 Sections are classified as Class 4 Sections. As shown in Figure 15 these sections buckle locally at moments less than M_y and their moment resistance must be expressed as a function of the width-to-thickness ratios of the plates composing the section.

Clause 13.5.1(c) divides Class 4 Sections into three categories. The first category contains those sections having both flange and web plates falling within Class 4. LSD and WSD Clauses 13.5.1(c)(i) requires that this type of section be designed to the requirements of Clause 14 using the material properties appropriate to the structural steel specified.

The second category contains those sections having flanges meeting the requirements of Class 3 but having webs sufficiently slender to place the section in Class 4. LSD and WSD Clauses 13.5.1(c)(ii) requires that this type of section be designed according to the requirements of Clause 16 which bases the moment resistance (LSD) or allowable bending stress (WSD) on a consideration of the redistribution of load carrying capacity between the portion of the slender web in compression, and the compression flange. These sections are generally referred to as plate girders.

Clause 13.5.1(c)(iii) treats those Class 4 Sections having web plates meeting the Class 3 requirements but slender compression flanges, falling within Class 4 limits. In this case, the moment resistance (LSD) or allowable bending stress

(WSD) is governed by local buckling of the compression elements. As an alternate to using Clause 14 to determine the critical stress for local buckling the designer may use an effective section modulus. The effective section modulus in turn is derived from a conservative interpretation of the effective widths given in Clause 14.

The value in LSD of $\phi = 0.90$ chosen for beams has been determined by a statistical analysis based on three series of tests where beams under a range of loading and restraint conditions were examined[45,112].

13.5.2 Bending - Laterally Unsupported Members

The strength of a laterally unsupported beam may be governed by lateral buckling of the member before the nominal bending capacity of the cross-section can be attained.

The resistance to lateral buckling depends upon the lateral bending stiffness of the cross-section (EI_y) as well as the resistance developed in pure (St. Venant) and warping torsion. The St. Venant torsional component is developed by the shear stresses in the individual plates making up the cross-section. The St. Venant stiffness is the product GJ where G represents the shear modulus of the material and J is the St. Venant torsional constant.

The warping resistance is generated by cross-bending of the flanges. As the beam twists the cross-section rotates about its centroidal axis and this motion induces lateral bending strains in the flanges. These strains result in the development of flange bending moments and accompanying shear forces. The couple produced by the shear forces makes up the warping torsional resistance and is a function of EI_y/C_w, where C_w is the warping torsional constant.

For a beam having simply-supported boundary conditions and prevented from twisting about its centroidal axis, the moment at which lateral buckling will occur is given by the equation in LSD Clause 13.5.2(a)(ii) and in WSD Clause 13.5.1(b)(ii). This expression assumes that the member is completely elastic at the moment corresponding to lateral buckling.

Limit States Design

If the bending moment corresponding to lateral buckling is greater than about 2/3 of the moment resistance for a laterally supported member, then the assumptions made in deriving the elastic buckling expressions are no longer valid as the compression flange has been considerably softened by yielding at the flange tips. This yielding is caused by the residual strains acting together with those due to the applied load. The moment resistance is therefore reduced according to an empirical equation if the value of M_u is above 2/3 M_p for Class 1 and 2 sections (LSD Clause 13.5.2(a)(i)) or 2/3 M_y for Class 3 and 4 sections (LSD Clause 13.5.2(b)(i)).

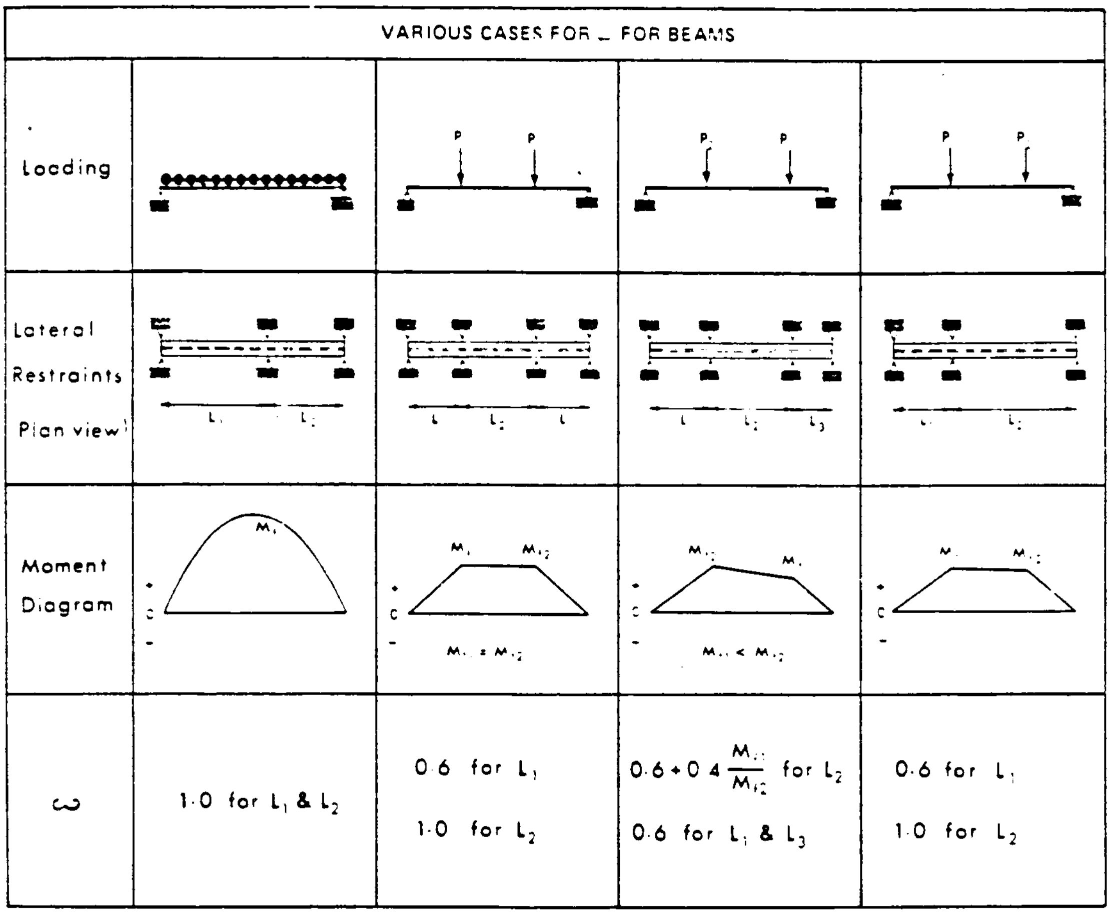
Regardless of the results of the lateral buckling calculations, in no case may the moment resistance exceed that based on the local buckling strength. Thus, $M_r < \phi\ M_p$ for Class 1 and 2 sections and $M_r < \phi\ M_y$ or $\phi\ SF_{cr}$ for Class 3 and 4 sections respectively.

The value in LSD of $\phi = 0.90$ chosen for laterally unsupported beams has been shown, by statistical analysis, to be generally conservative except for elastic buckling of rolled W shapes where $\phi = 0.84$ (7 percent less) was determined[112]. Since the actual end conditions of a beam in a structure will provide some degree of increased restraint over the laboratory situation, and to avoid the complexities associated with a variable ϕ value, the constant value in LSD of $\phi = 0.90$ is used for this type of member.

The requirements of this clause may well be conservative due to the assumptions regarding boundary conditions. Although the requirements specify that L is the length of the beam between lateral supports, it is logical in some cases to take L as the distance between points of contraflexure on the laterally buckled shape of the compression flange. In other cases, however, when yielding of adjacent spans may have reduced the restraint, this procedure is not recommended. In view of the difficulty in assessing the various situations, this Standard does not specify any adjustment for end restraint.

The LSD Standard does permit the use of an equivalent moment factor, ω, for use when the bending moment is not uniform over the length of the member. The value of ω used is the same as that for beam-columns. The value of ω determined by Clause 13.8.2(a)(i) is applicable only when the moment gradient for the applied loads is linear between the points of lateral support for which the values of M_{f_1} and M_{f_2} are appropriate. Thus, for the common case of a beam supporting a uniformly distributed load, ω always has the value of 1.0. Figure 17 illustrates selected

Figure 17



cases of ω for various loading and support conditions. Reference 25 recommends values of ω for unusual loading and support conditions.

For I-shaped members the various cross-sectional properties may be approximated (by neglecting the contribution of the web) and M_u can be expressed in terms of parameters σ_1 and σ_2[22].

Channels prevented from twisting are treated in a manner similar to I-sections except that the lateral flange bending component σ_2 is assumed to be zero.

Working Stress Design

For I-type sections the elastic lateral buckling stress F_1 is the square root of the sum of the squares of two components F_2, the St. Venant torsional component and F_3, the lateral flange bending component. F_2 and F_3 have been divided by factors of safety of 1.67 and 1.92 respectively. If F_1 thus computed is $\leq 2/3\ F_{bt}$, the allowable tensile bending stress, it is a valid approximation of the true buckling stress with a margin of safety of at least 1.67. However, if F_1 is $> 2/3\ F_{bt}$ it is no longer a valid approximation due to inelastic effects and must be reduced in a manner similar to that recommended by the Column Research Council. In no case should the allowable compressive bending stress F_{bc} exceed the allowable tensile bending stress F_{bt}. For closely or continuously braced beams, no reduction in strength occurs and $F_{bc} = F_{bt}$. The same formulas apply to both compact and non-compact sections since in the elastic range the degree of compactness is not a determining factor while in the inelastic range the introduction of F_{bt} into the formula affords the required differentiation since F_{bt} is higher for compact than for non-compact sections.

Channels prevented from twisting are treated in a manner similar to non-compact I-sections except that the lateral flange bending component F_3 is assumed to be zero.

In WSD Clause 13.5.2 for hollow structural sections, F_{bc} is not reduced from the basic value F_b provided that the unsupported length does not exceed $17230/F_y$ times the width of the section.

Tee sections having an axis of symmetry in the plane of bending are permitted to be proportioned for the basic bending stress F_b provided lateral support is provided at intervals not exceeding 630 $r_y/\sqrt{F_y}$. Since this limit is based on the local buckling strength the orientation of the tee is not a factor.

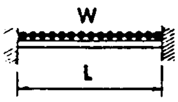
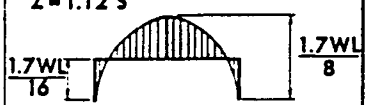
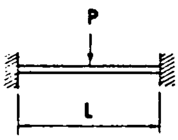
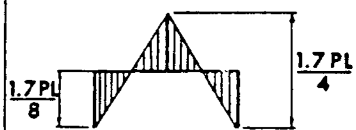
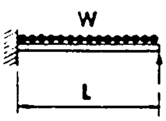
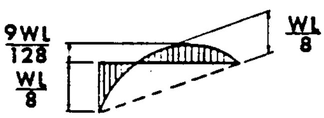
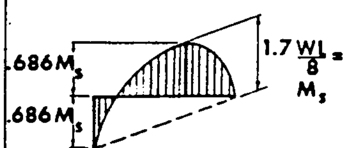
13.5.5 (WSD) Clause 13.5.5 in WSD permits continuous and rigidly framed beams supporting gravity loads to be designed on a special basis provided that they meet the requirements for plastic design sections as specified in Clause 31. In fact Clause 13.5.5 is basically a plastic design clause written on an allowable stress basis and for this reason the width-thickness limits must be more stringent in order to provide the necessary rotational capacity.

Figure 18 illustrates the design method for three different beams and provides the plastic design solutions in accordance with Clause 31 using a load factor of 1.7. In general it is preferable to design continuous beams according to the requirements of Clause 31 if plastic design sections are used. On the other hand, some designers prefer to compute bending moments by elastic theory and only for specified loads. Clause 15.5.5 (WSD) permits them to do so and take advantage of most of the extra load carrying capacity offered by continuous beams with sturdy width-thickness ratios.

Figure 18

CONTINUOUS BEAMS MEETING REQUIREMENTS

FOR PLASTIC DESIGN SECTIONS

Example	Proportioned by Clause 13.5.6	Proportioned by Clause 31
	 $S_{req} = \frac{WL}{8}/1.32\ Fy = \frac{WL}{10.56\ Fy}$ $S_{req} = \frac{WL}{12}/0.75\ Fy = \frac{WL}{9.0\ Fy}$ $\therefore$ $WL/9.0\ F_y$ Governs	Z = 1.12 S  $Z_{req} = 1.7\frac{WL}{16}/Fy = \frac{WL}{9.41\ Fy}$ $S_{req} = \frac{WL}{1.12 \times 9.41\ Fy} = \frac{WL}{10.54\ Fy}$
	 $S_{req} = \frac{PL}{4}/1.32\ Fy = \frac{PL}{5.28\ Fy}$ $S_{req} = \frac{PL}{8}/0.75\ Fy = \frac{PL}{6.0\ Fy}$ Greater Value, $\frac{PL}{5.28\ Fy}$, Governs	 $Z_{req} = 1.7\frac{PL}{8}/Fy = \frac{PL}{4.71\ Fy}$ $S_{req} = \frac{PL}{1.12 \times 4.71\ Fy} = \frac{PL}{5.28\ Fy}$
	 $S_{req} = \frac{WL}{8}/0.75\ Fy = \frac{WL}{6.0\ Fy}$	 $Z_{req} = .686 \times 1.7\frac{WL}{8}/Fy = \frac{WL}{6.86\ Fy}$ $S_{req} = \frac{WL}{1.12 \times 6.86\ Fy} = \frac{WL}{7.68\ Fy}$

13.6 Lateral Bracing for Members in Structures Analysed . Plastically

See the Commentary on Clause 8.5(c).

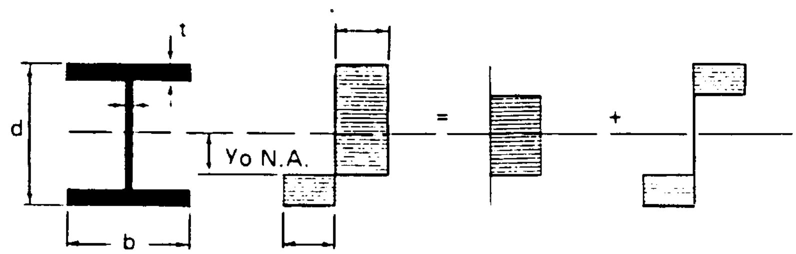
13.7 Axial Compression and Bending

There are three requirements which must be met to ensure the adequacy of any beam-column. As prescribed by Clause 11.3 of the Standard, the compression elements of the cross-section must be proportioned such that local buckling does not occur prior to attainment of the cross-section strength. In Clause 13.7, requirements relating to a strength check of the cross-section and requirements relating to a stability check of the entire member are set forth.

13.7.1 Clause 13.7.1(a) is a statement of the principle involved in the strength check. It may be applied simply as follows. Using LSD consider a W-shape subject to moment, M_f, and thrust, C_f (Figure 19). If part of the web, taken at yield stress, will just sustain the thrust, C_f, the remainder of the cross-section (flanges plus adjoining web) is available to resist the moment, M_f, at yield stress. Such a principle is relatively easy to apply for uniaxial bending. For W shapes, References 12 and 46 give explicit expressions for the plastic moment capacity, reduced for the presence of axial load. In working stress design the maximum stress is limited to a fraction of the yield strength. This approach is not as satisfactory as the LSD approach.

Figure 19

IDEALIZED STRESSES IN PLASTIFIED SECTION OF BEAM-COLUMN



(a) Total stress distribution (b) Stress due to C_f (c) Stress due to M_f

For biaxial bending, no simple methods exist for an exact strength check, since both the location and angular disposition of the neutral axis have to be determined. For this reason, interaction expressions have been introduced to predict member strength. Their accuracy tends to be dependent on the complexity of the expression and the shape of the cross-section.

Clause 13.7.1(b) is the general form of the stability interaction expression, applicable to any compact shape. Semi-empirical in origin, it is formula 8.29 from Reference 25.

The equivalent moments used in the numerator terms are discussed under Clauses 13.7.4 (a), (b) and (c). Amplification factors $(1-C_f/C_e)$ are included in the denominator bending terms to allow for the additional bending caused by the axial load acting in the deformed column. The major axis resisting moment or bending stress reflects the propensity of the member to fail through lateral-torsional buckling.

13.7.2 (LSD) Clauses 13.7.2(a) and (b) in LSD are strength checks for I and W Class 1 or 2 shapes. Clause 13.7.2(a) controls when bending is dominant. Reference 12 gives the following expressions for the plastic moment of resistance, reduced for the presence of axial loads and are shown in Figure 20.

$$M_{rcx} = 1.18\ M_{rx}\ (1-C_f/C_y) < M_{rx}$$

$$M_{rcy} = 1.19\ M_{ry}\ [1-(C_f/C_y)^2] < M_{ry}$$

(These expressions are used in Appendix L 2.7A - Limit States Design in which more refined approaches to biaxial bending are developed).

For bending about the X-X axis, the expression may be transposed as follows:

$$\frac{C_f}{C_y} + 0.85\ \frac{M_{rcx}}{M_{rx}} < 1.0$$

The following linear expression has been suggested as a good approximation to the second-order for M_{rcy}[47];

$$\frac{C_f}{C_y} + 0.6\ \frac{M_{rcy}}{M_{ry}} < 1.0$$

LSD Clause 13.7.2(b) results by combining these expressions.

stressed corner of the cross-section, to the nominal yield stress. In fact, because of residual stresses, some yielding may occur at such a fibre if this expression is just satisfied. For Class 4 sections, the stress limit may be less than nominal yield depending on the critical stresses for the particular section.

Again, Clause 13.7.3(b) is an empirical linear interaction expression relating the critical axial load ratio and the ratios of applied moment to limiting moment capacity about the two orthogonal axes.

13.7.4 (LSD) and 13.7.2 (WSD) This clause lists values of ω a coefficient used to compute a uniform bending effect in beam-columns which is intended to be equivalent to the effect of a non-uniform moment on the stability of the member. This coefficient is explained in more detail in Reference 25 where it is called C_m. Figures 21 and 22 offer guidance in the LSD design of beam-columns subjected to various bending moment effects. Figures 23 and 24 provide the same information for the WSD design of beam columns. Reference 48 contains more guidance on the design of beam-columns subject to concentrated load (or moment) between supports.

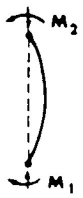
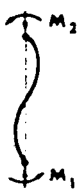
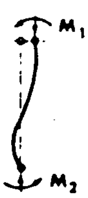
13.8 Axial Tension and Bending

A simple, generally conservative, interaction formula is given in LSD and WSD Clauses 13.8(a) to check the adequacy of members subjected to combined bending and axial tension.

The equation given in LSD and WSD Clauses 13.8(b) is to guard against the case when the tensile force is relatively small

Figure 21

GUIDE TO DESIGN OF BEAM-COLUMNS
PRISMATIC MEMBERS – MOMENT AT ENDS – NO TRANSVERSE LOADS

Conditions		P–Δ Effects Included	ω	Design Criteria: Class 1 and 2 I-shape columns	Design Criteria: Class 1 and 2 sections	Design Criteria: Class 3 and 4 sections
	$M_2 \geq M_1$ Single curvature bending	YES	$0.6 + 0.4\frac{M_1}{M_2}$			
		NO	1.0			
	$M_1 = 0$ Single curvature bending	YES	0.6	$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$ $\frac{C_f}{C_r} + 0.85\frac{M_{fx}}{M_{rx}} + \frac{0.60 M_{fy}}{M_{ry}} \leq 1.0$	$\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$	$\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$
		NO	0.85	$\frac{C_f}{C_r} + \frac{\omega_x M_{fx} U_x}{M_{rx}} + \frac{\omega_y M_{fy} U_y}{M_{ry}} \leq 1.0$	$\frac{C_f}{C_r} + \frac{\omega_x M_{fx} U_x}{M_{rx}} + \frac{\omega_y M_{fy} U_y}{M_{ry}} \leq 1.0$	$\frac{C_f}{C_r} + \frac{\omega_x M_{fx} U_x}{M_{rx}} + \frac{\omega_y M_{fy} U_y}{M_{ry}} \leq 1.0$
	$M_2 \geq M_1$ Double curvature bending	YES	$0.6 - 0.4\frac{M_1}{M_2}$ $\nless 0.4$			
		NO	0.85			

C_f = Factored compressive load
C_r = Factored compressive resistance
M_f = Factored bending moment (x-x or y-y axis)
M_r = Factored moment resistance (x-x or y-y axis)
ω = coefficient used to determine equivalent uniform bending effect (X-X or y-y axis)
U = Amplification factor = $\frac{1}{1 - (C_f/C_e)}$ (x-x or y-y axis)

Figure 22

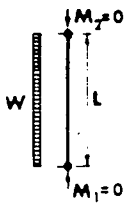
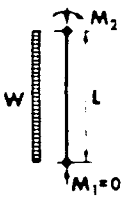
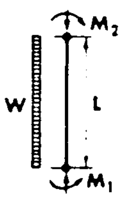
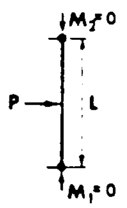
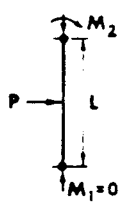
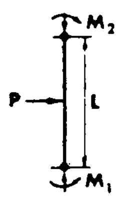
GUIDE TO DESIGN OF BEAM-COLUMNS PRISMATIC MEMBERS – TRANSVERSE LOADS				
CONDITIONS		M_f	ω*	DESIGN CRITERIA
	Load W distributed uniformly. $M_2 = M_1 = 0$ Max. span moment $M_3 = \frac{WL}{8}$	M_3	1.0	For Class 1 and 2 I-shaped columns $\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leqslant 1.0$
	Load W distributed uniformly. $M_2 = \frac{WL}{8}$ $M_1 = 0$ Max. span moment $M_3 = \frac{9WL}{128}$	M_2	1.0	$\frac{C_f}{C_r} + \frac{0.85\ M_{fx}}{M_{rx}} + \frac{0.60\ M_{fy}}{M_{ry}} \leqslant 1.0$ $\frac{C_f}{C_r} + \frac{\omega_x\ M_{fx}\ U_x}{M_{rx}} + \frac{\omega_y\ M_{fy}\ U_y}{M_{ry}} \leqslant 1.0$
	Load W distributed uniformly. $M_2 = M_1 = \frac{WL}{12}$ Max. span moment $M_3 = \frac{WL}{24}$	M_2	1.0	For Class 1 and 2 sections
	Load P at mid-span. $M_2 = M_1 = 0$ Max. span moment $M_3 = \frac{PL}{4}$	M_3	0.85	$\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leqslant 1.0$ $\frac{C_f}{C_r} + \frac{\omega_x\ M_{fx}\ U_x}{M_{rx}} + \frac{\omega_y\ M_{fy}\ U_y}{M_{ry}} \leqslant 1.0$
	Load P at mid-span. $M_2 = \frac{3PL}{16}$ $M_1 = 0$ Max. span moment $M_3 = \frac{5PL}{32}$	M_2	0.85	For Class 3 and 4 sections $\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leqslant 1.0$
	Load P at mid-span. $M_2 = M_1 = \frac{PL}{8}$ Max. span moment $M_3 = \frac{PL}{8}$	M_2	0.85	$\frac{C_f}{C_r} + \frac{\omega_x\ M_{fx}\ U_x}{M_{rx}} + \frac{\omega_y\ M_{fy}\ U_y}{M_{ry}} \leqslant 1.0$
Note: See Clause 13.7.4(c) for alternate procedure for concentrated load cases. *See Article 8.6 of reference 25 for alternate methods of determining ω, (Cm).				

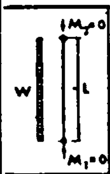
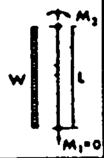
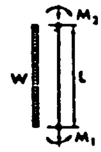
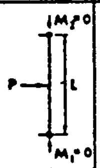
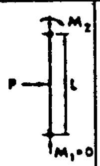
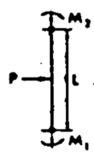
Figure 23

GUIDE TO DESIGN OF BEAM-COLUMNS PRISMATIC MEMBERS — MOMENT AT ENDS — NO TRANSVERSE LOADS				
CONDITIONS		f_b	ω	DESIGN CRITERIA
	$M_2 > M_1$ Joint translation prevented. Single curvature bending.	M_2/S	$0.6 + 0.4 \frac{M_1}{M_2}$	$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} < 1$ $\frac{f_a}{F_a} + \frac{\omega f_b}{F_b(1 - \frac{f_a}{F'_e})} < 1$
	$M_1 = 0$ Joint translation prevented. Single curvature bending.	M_2/S	0.6	
	$M_2 > M_1$ Joint translation prevented Double curvature bending.	M_2/S	$0.6 - 0.4 \frac{M_1}{M_2}$ $\nless 0.4$	
	$M_2 > M_1$ Joint translation not prevented. Single curvature bending.	M_2/S	1.0	$\frac{f_a}{F_a} + \frac{\omega f_b}{F_b(1 - \frac{f_a}{F_e})} < 1$
	$M_1 = 0$ Joint translation not prevented Single curvature bending.	M_2/S	0.85	$\frac{f_a}{0.6F_y} + \frac{f_b}{F_{bo}} < 1$ $\frac{f_a}{F_a} + \frac{\omega f_b}{F_b(1 - \frac{f_a}{F'_e})} < 1$
	$M_2 > M_1$ Joint translation not prevented Double curvature bending.	M_2/S	0.85	
f_a = Computed axial stress. F_a = Allowable axial stress. f_b = Computed max. bending stress due to primary moments. = Coefficient to determine eqivalent uniform bending stress.			F_b = Allowable bending stress between supports. F_{bo} = Allowable bending stress at support. S = Section modulus of beam-column.	

Figure 24

GUIDE TO DESIGN OF BEAM-COLUMNS

PRISMATIC MEMBERS — TRANSVERSE LOADS — JOINT TRANSLATION PREVENTED

CONDITIONS		f_b	ω	DESIGN CRITERIA
	Load W distributed uniformly $M_2 = M_1 = 0$ Max span moment $M_3 = \frac{WL}{8}$	M_3/S	1.0	$\frac{f_a}{F_a} + \frac{\omega f_b}{F_b(1 - \frac{f_a}{F'_e})} < 1$
	Load W distributed uniformly $M_2 = \frac{WL}{8}$ $M_1 = 0$ Max. span moment $M_3 = \frac{9WL}{128}$	M_2/S	$1-.3\frac{f_a}{F'_e}$	$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} < 1$ $\frac{f_a}{F_a} + \frac{\omega f_b}{F_b(1 - \frac{f_a}{F'_e})} < 1$
	Load W distributed uniformly. $M_2 = M_1 = \frac{WL}{12}$ Max. span moment $M_3 = \frac{WL}{24}$	M_2/S	$1-.4\frac{f_a}{F'_e}$	
	Load P at mid-span $M_2 = M_1 = 0$ Max. span moment $M_3 = \frac{PL}{4}$	M_3/S	$1-.2\frac{f_a}{F'_e}$	$\frac{f_a}{F_a} + \frac{\omega f_b}{F_b(1 - \frac{f_a}{F'_e})} < 1$
	Load P at mid-span $M_2 = \frac{3PL}{16}$ $M_1 = 0$ Max. span moment $M_3 = \frac{5PL}{32}$	M_2/S	$1-.4\frac{f_a}{F'_e}$	$\frac{f_a}{0.6F_y} + \frac{f_b}{F_{bo}} < 1$ $\frac{f_a}{F_a} + \frac{\omega f_b}{F_b(1 - \frac{f_a}{F'_e})} < 1$
	Load P at mid-span $M_2 = M_1 = \frac{PL}{8}$ Max. span moment $M_3 = \frac{PL}{8}$	M_2/S	$1-.6\frac{f_a}{F'_e}$	

Note: See clause 13.7.2 for alternate procedure for concentrated load cases.

with the result that the compression flange remains in compression and therefore the capacity of the section is still controlled by the capacity of the compression flange.

13.9 Load Bearing

The bearing resistance (LSD) or allowable bearing stress (WSD) given for machined, accurately sawn or fitted part in contact (Clause 13.9(b)) reflects the fact that a triaxial compressive stress state generally exists which restricts yielding of the parts in contact.

For a cylindrical roller or rocker in contact with a flat surface, the maximum shearing stress developed due to a line load of q kN/mm is

$$\tau_{max} = 0.27 \frac{qE}{\pi D(1-\nu^2)}$$

where ν is Poisson's ratio[49]. From this the bearing resistance (LSD) is then

$$\frac{B_r}{\phi} = qL = \frac{\pi DL(1-\nu^2)(\tau_{max})^2}{0.27^2 E}$$

Setting $\tau_{max} = \frac{4}{3}\tau_y$ and $\tau_y = \frac{F_y}{\sqrt{3}}$ leads to

$$\frac{B_r}{\phi} = \frac{\pi DL(1-\nu^2)}{0.27^2 E} \frac{4}{3} \times \frac{F_y}{\sqrt{3}} = 0.00013DLF_y^2$$

The "Hertz" solution, as reported in Reference 50 gives the allowable load as

$$2.86DL \frac{(2.7F_y)^2}{E} = 0.00010DLF_y^2$$

and indicates that the value of 0.00013 obtained by calibration with the earlier WSD Standards for a yield stress of about 400 MPa is somewhat conservative.

In the WSD Standard for the allowable compressive bearing stress $F_p = 0.0008F_y^2$, the coefficient 0.00008 reflects the application of a nominal factor of safety of 5/3.

For bearing-type connections, LSD and WSD Clauses 13.9(c), tests have shown[51,52,53,54] that the ratio of the bearing stress (B_r/dt in LSD or F_p in WSD) to the ultimate tensile strength of the plate (F_u) is in the same ratio as the end distance of the bolt (e) to its diameter (d). Thus,

$$\frac{B_r}{dtF_u} = \frac{e}{d}$$

or, for n fasteners, $B_r = tneF_u$

As the test results do not provide data for e/d greater than 3, an upper limit of e = 3d is imposed, that is

$$B_r \leqslant 3tdnF_u$$

As for connections in general, the value of ϕ in the LSD Clause 13.9(c) is to be taken as 0.67. This value has been reduced from 0.90 as a result of a pilot series of tests which indicated that lower failure loads could occur in thin webs of coped beams with compact connections[115]. (See also the Commentary to Clause 13.10.)

13.10 Bolts in Bearing-Type Connections

13.10.1 It is the initial premise to the resistances developed in this clause that the strength of the structure should be governed by member capacity rather than by that of the connector. Hence, the value of ϕ to be used in LSD is established at a lower value (0.67) than that prescribed for members (0.90). This is applicable to all parts of Clause 13.10. Similarly, the factor of safety used in WSD is established at a higher value (2.5) as compared to that for members (1.67).

13.10.2 The strength of a bolted connection can be governed by the bearing capacity of the material abutting the fastener (Clause 13.9(c)), by the net section capacity if in tension (Clause 13.2(a)), or by the shear strength of the fasteners. The latter is the subject of Clause 13.10.2.

Based on extensive testing, it has been established that the shear strength of high-strength bolts is approximately 0.60 times the tensile strength of the bolt material. Hence, to obtain the shear resistance in LSD of a group of bolts, this quantity is multiplied by the cross-sectional area of one bolt, the number of shear planes in the joint, and the total number of bolts resisting the load ($V_r = 0.60\ \phi\ n\ m\ A_b\ F_u$).

Two modifications are necessary in special circumstances. If the bolt thread is intercepted by a shear plane, there is less shear area available than that given above. The ratio of the area through the thread root of a bolt to its shank area is about

0.70 for the usual structural sizes. (In unusual cases, such as thin parts and small bolt diameters, the threaded part of the shank may be intercepted by two shear planes. The designer should further modify the shear strength equation accordingly.)

The second possible modification concerns joint length. It has been well established that, except for the case of two bolts in line, joint strength is not linearly proportional with joint length. The average resistance per fastener decreases with joint length. In the interest of simplicity, the Standards break down joint strength into two cases. Joints less than 1300 mm long require no reduction when calculating the total shear resistance (LSD) or allowable shear stress (WSD) of the bolts while those greater than 1300 mm long are to be taken as 80% of the basic value. This "step" evaluation provides a reasonable approximation to the true case.

13.10.3 It is intended that the designer include an estimate of the force (LSD) or stress (WSD) that may be present due to prying action when calculating the tensile force (LSD) or tensile stress (WSD) present in a member. The "Guide to Design Criteria for Bolted and Riveted Joints"[55] gives several recommendations as to how to obtain this. If the connection is subjected to repeated loading, prying action must be avoided.

In the LSD Standard the ultimate resistance of a single high-strength bolt loaded in tension by the connected parts is equal to the product of its stress area (a value lying between the gross bolt area and the area taken through the thread roots) and the ultimate tensile strength of the bolt material. For simplicity, the equation given in the LSD Clause 13.10.2 uses the nominal area of the bolt (A_b) and the multiplier 0.75 to provide an approximate conversion to the stress area.

13.10.4 The expression given in this clause is an elliptical interaction equation developed directly from test results. In LSD the value 0.56 is the square of 0.75, the necessary conversion from nominal bolt area to stress area and the empirical parameter, β, is dependent upon bolt type and location of the shear plane with respect to the threads.

13.11.2 Bolts in Slip-Resistant (Friction-Type) Connections

In the design of a slip-resistant joint, the first concern is the probability of slip under specified loads. The load transfer is by friction developed between the faying surfaces of the joint. This frictional force is developed as a consequence of the high clamping force resulting from the tightening of the bolts. However, the ultimate strength of the bolted joint, after slip into bearing has occurred, is still dependent upon the bearing and shear resistances as a bearing-type joint.

The slip resistance of a bolted joint depends upon the number of faying surfaces, the coefficient of friction between the parts being joined, and the total clamping force provided by the bolts. In addition to these quantities, it is recognized that the ideal situation of zero percent probability of slip is not attainable. The designer must choose the slip probability level that he thinks appropriate to the structure being considered.

Both the slip coefficient and the initial clamping force have considerable variation about their mean values. The necessary frequency distributions for these effects are known for a large number of practical cases and they have been used to evaluate the slip probability levels for various situations.

The values of μ to be used in the equations of Clauses 13.11.2 to give the slip resistance V_s in LSD and the allowable stress, F_v in WSD are given in Tables 4 of the Standards. These values presented in Tables 4 combine the effects of the probable clamping force and the type and condition of the faying surfaces and have been chosen for the 5% probability level, that is, there is a 5% chance that the joint will slip into bearing under the specified loads. For connections which are desired to be slip resistant but for which a larger probability of slip is tolerable, a table of μ-values for the 10% level for use with LSD, is available in Reference 55.

The numerical modifier in the equation given in LSD Clause 13.11.2 (that is. 0.26) includes the necessary relationship between tensile area and nominal bolt area and the relationship between bolt tensile strength and required proof load. It also contains a component that enables the use of the equation as given along with published values of μ in Reference 55. In this reference, they are used in conjunction with shear stress values. 2.7A - Limit States Design avoids this since bolts in a friction-type connection are, by definition, never acting in shear at specified load levels.

Designers are reminded that the use of high-strength bolts in a friction-type connection should be the exception rather than the rule. They are the preferred solution where cyclic loads or load reversals are present or where the use of the building is such that the small slips that might otherwise occur cannot be tolerated.

13.11.3 Connections in Combined Shear and Tension This clause considers the case of a friction-type shear connection which also has a component of load parallel to the axes of the bolts.

Starting with the case where there is no component of load parallel to the axis of the bolts the resistance to slip will be equal to the full value. As tensile load is applied the resistance to slip will be reduced and will equal zero when the parts are on the verge of separation. Assuming that the clamping force reduces in proportion to the applied tension the interaction relationship will be linear. (See also Clause 23.1.5.)

In the LSD equation let the applied tension be T, the applied shear be V and the initial tension be T_i. The shear that can be carried when the tension T_i is zero is:

$$V_s = K_s T_i$$

where K_s is the coefficient of friction.

Therefore, the shear, V, that can be carried when the clamping force is reduced because of the tension T is:

$$V = K_s\ (T_i - T)$$

$$V = K_sT_i - K_sT$$

$$= V_s - K_sT$$

$$\frac{V}{V_s} = 1 - \frac{K_sT}{V_s}$$

$$\frac{V}{V_s} + \frac{K_s\ T}{K_s\ T_i} = 1$$

but $T_i = 0.70 \times 0.75\ A_bF_un = \dfrac{n\ A_b\ F_u}{1.90}$

$$\therefore \frac{V}{V_s} + \frac{1.90\ T}{n\ A_b\ F_u} = 1$$

Because the ultimate tensile strength is about 2.05 times the allowable tensile stress when both are determined on the stress area of the fastener the above expression becomes:

$\dfrac{f_v}{F_v} + 0.93\ \dfrac{f_t}{F_t} \leq 1.0$ which can conservatively be taken as

$$\frac{f_v}{F_v} + \frac{f_t}{F_t} \leq 1.0$$

13.12 Welds

The major area of consideration is that of the shear resistance (LSD) or stress (WSD) of complete or partial penetration groove welds, plug and slot welds, and of fillet welds. The resistances (LSD) or allowable stresses (WSD) of welds in other categories (tension or compression parallel to axis of complete or partial penetration groove welds and or fillet welds, tension or compression normal to the throat of complete groove welds, and compression normal to the throat of partial penetration groove welds) are taken as the same as those for the base metal. Tension other than parallel to the axis of partial penetration groove welds is considered to be the same as shear.

The Standard explicitly recognizes that the shear resistance (LSD) or allowable shear stress (WSD) of a weld must be evaluated on the basis of both the resistance of the weld itself and of the base metal adjacent to the weld.

The resistance of the base metal in LSD is given as $V_r = 0.66\ \phi\ A_m\ F_y$ ($0.40\ F_y$ in WSD). This expression is consistent with that given in the LSD Clause 13.4.1(a) for the shear resistance of a flexural member with a stocky web. The area of metal (A_m) to be used here is the area of the fusion face. The shear yield of steel is customarily taken to be $F_y/\sqrt{3}$, that is, $0.58\ F_y$. The increase between this value and that given in the equation above ($0.66\ F_y$) is attributable to the beneficial effects of strain-hardening. The value of ϕ to be used in evaluating the shear resistance of the base metal will normally be taken as 0.90.

The strength of the weld metal in LSD is given as $V_r = 0.67\ \phi_w\ A_w\ X_u$ ($0.30\ X_u$ in WSD). The term A_w is the effective throat area of the weld and X_u is the ultimate tensile strength of the electrode (as given by the electrode classification number).

As has already been stated, both the fastening element (the weld) and the connected material are considered in this clause. As was noted for high-strength bolts, it is desirable to ensure that the fasteners will not fail before the members. In that case, ϕ is taken as 0.67 and is also used in the equation for fastener resistance given in this section. For a load factor of 1.5, the results obtained here will be comparable to those in WSD.

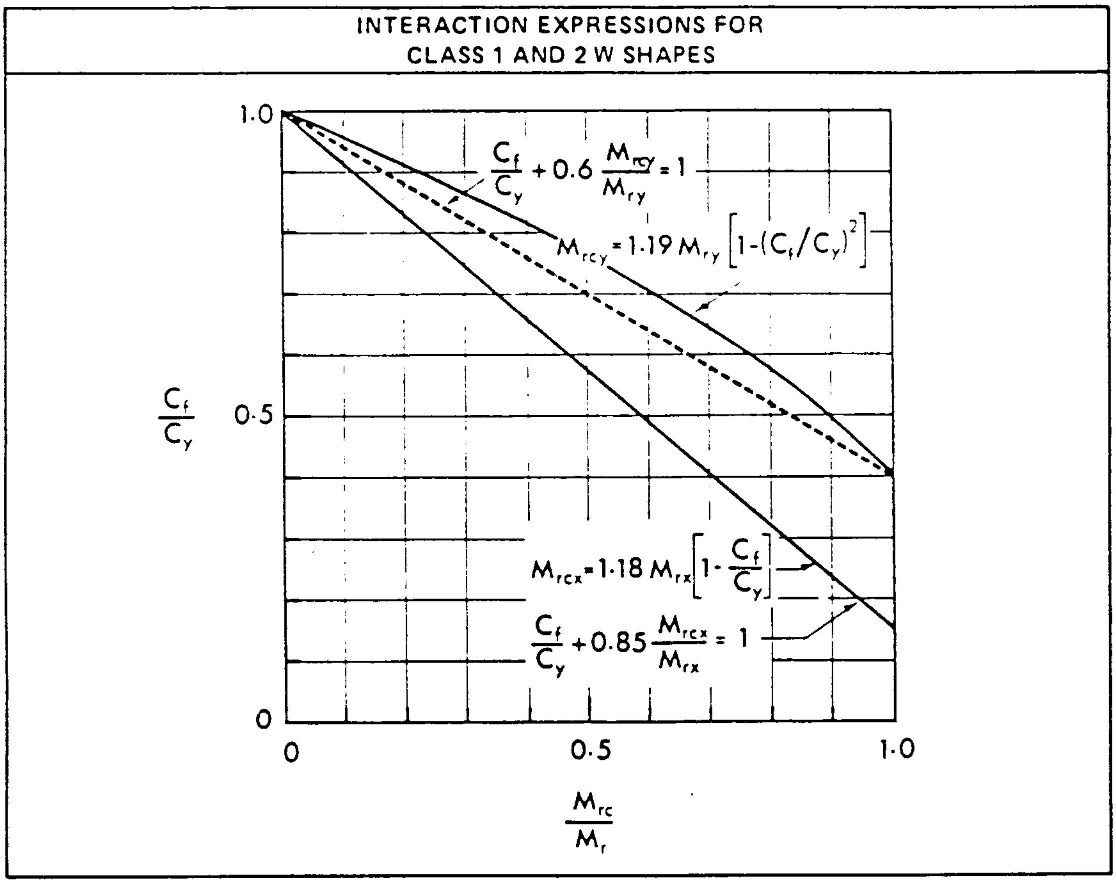
14. Cold Formed Steel Structural Members

14.1 Scope

Clause 14 applies first and foremost to the design of cold formed steel structural members. Such members are roll or brake formed from carbon or low alloy flat rolled steel products. Cold formed members find application where hot rolled shapes would be uneconomical, where cross sections are required which are not feasible by hot rolling or by other methods of fabrication, and in particular where large useful surfaces as in deck and cladding are needed. Cold formed steel structural members may be broadly divided into two categories (a) individual structural sections, for example channels, zees, hats; and (b) large-surface elements, for example deck, cladding, panels.

Cold formed components may also be combined with hot rolled components in a single entity such as an open web steel joist

Figure 20



LSD Clause 13.7.2(c) is identical in form to that of LSD Clause 13.7.1(b). For W and I shapes, the shape factors inherent in M_{rx} and M_{ry} are about 1.12 and 1.55, respectively. In the Commentary to LSD Appendix L, it is demonstrated that the economies can be achieved by using slightly more complex non-linear interaction expressions.

13.7.3 (LSD) For Class 3 sections LSD Clause 13.7.3(a) essentially limits the extreme fibre stress, at the most highly having cold formed chords and hot rolled web members. In such cases the general design requirements for the basic product may be covered by other clauses and Clause 14 would be used in conjunction with the general design requirements to effect a proper design of the cold formed elements.

Clause 14 also applies to the design of steel structural members covered by other clauses in cases where the width-thickness ratio of any component of a member, whether or not cold formed, exceeds that normally allowed. In this application, Clause 14 is supplementing, not supplanting the basic design standard.

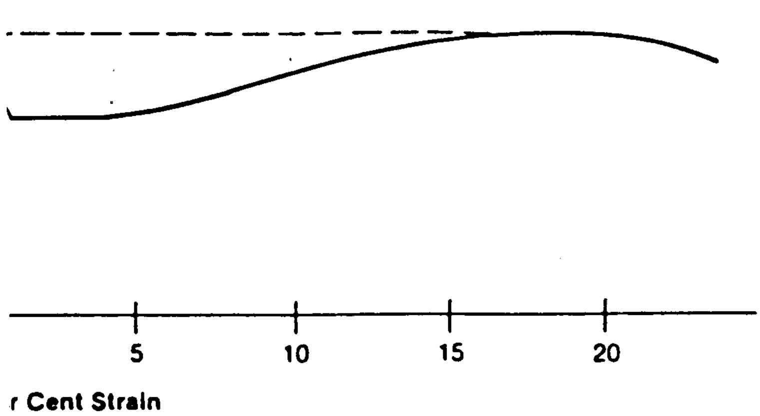
The bulk of experimental evidence supporting the provisions of Clause 14 has been obtained on specimens with material thickness between 0.38 and 6.35 mm with some recent work on material up to 25 mm in thickness.[142] No significant influence of thickness has been observed and the requirements of this clause may be deemed to apply to members of any thickness capable of being cold formed, except in the case of connections (see Clause 14.6) where certain requirements such as shear resistance (LSD) or allowable shear stresses (WSD) in welds, and bearing resistance (LSD) or allowable bearing stresses (WSD) exerted by mechanical fasteners are noted to apply only to a specific range of thickness. Where the requirements of Clause 14 are being applied in conjunction with and supplementary to the requirements of another clause any upper limitation on thickness would be based on the requirements of the clause would normally govern.

Clause 14 differs from other clauses, in part, in that the process of cold forming induces selective strain hardening at corners which affects response to load in a manner quite different from the response of hot rolled members; and Clause 14 allows this to be taken into account. But equally important, in contrast to individual structural sections whose prime function is to carry load, the structural strength of many cold formed members such as deck, cladding, and various members of pre-engineered steel building systems is only one of several desired functions. The optimum shape or profile is therefore not necessarily the one which would be chosen from structural considerations alone. In particular the width thickness ratio of flat elements may be well in excess of that which would be structurally economical. In order to utilize the full strength of such elements it is necessary to take into account post-buckling strength. Designers of hot rolled steel construction, by contrast, are rarely concerned with post-buckling strength because the design standards impose width thickness limits which are intended to preclude element buckling prior to overall member buckling.

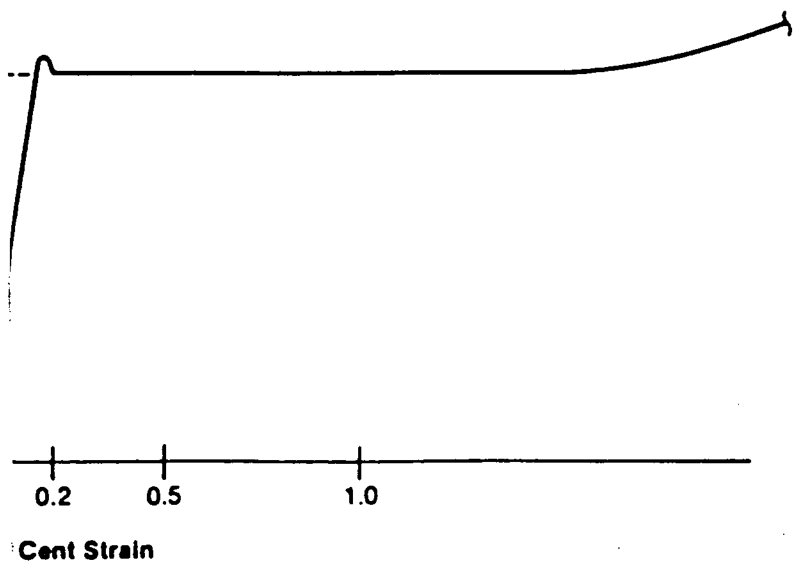
14.3 Material Standards

Two general types of tensile stress-strain curves exist for carbon and low alloy steel products. The sharp-yielding type shown in Figure 25 occurs for hot rolled steels and for some cold formed steels subject to strain-aging after strain-hardening. The slope of the stress-strain curve is essentially equal to the

Figure 25



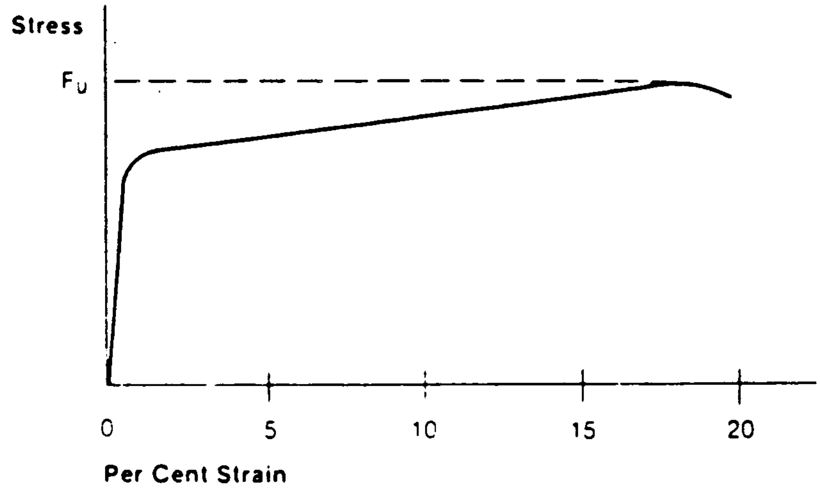
Stress-Strain Diagram



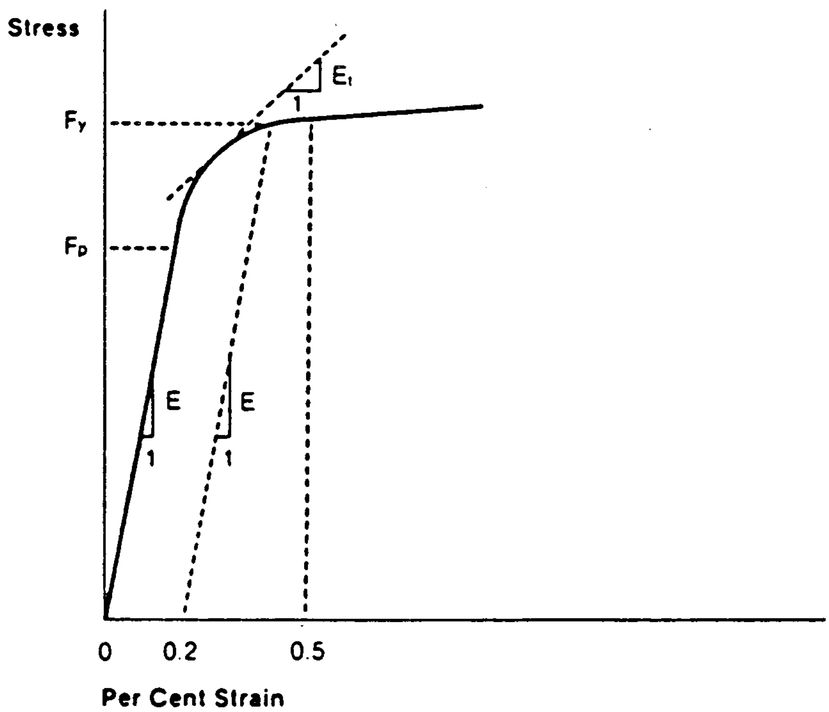
Initial Region of Stress-Strain Diagram Enlarged

asticity up to the yield strength since the mit is virtually equal to the yield strength.

Figure 26



Stress-Strain Diagram



Initial Region of Stress-Strain Diagram Enlarged

The gradual-yielding curve of Figure 26 is typical of most cold formed steels. The proportional limit is lower than the yield strength. Yield strength is usually defined by a 0.2 per cent offset strain or by a total elongation under load of 0.5 per cent as shown in the Figure. These two criteria normally produce similar results. Above the proportional limit, the tangent modulus is variable and less than the modulus of elasticity.

Mill Tests. Standard mill tests to determine certified mechanical properties including yield strength are based on tensile coupons since compression tests on coupons are very expensive for quality control purposes. It is also difficult to obtain accurate results from compression coupon tests. Carefully conducted compression coupon tests have shown that the compressive yield strength for a coupon is comparable to the tensile yield strength. Therefore, the yield strength (F_y) used in this clause is the tensile coupon yield strength.

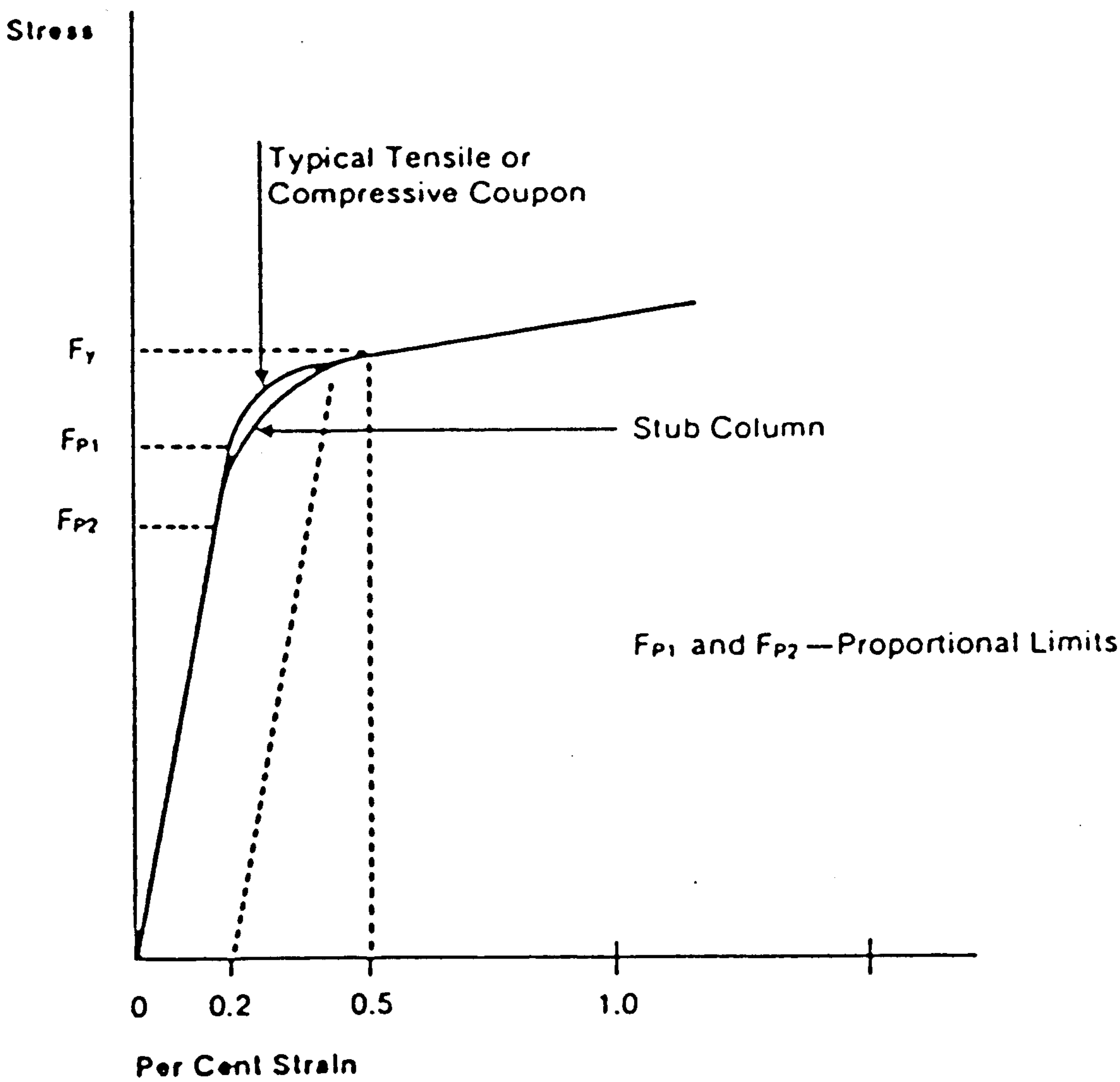
Design in Tension. For the various parts of Clause 14 which depend on tensile properties, the acceptance of tensile coupon tests is readily explained. As shown in Figure 26, the proportional limit for gradual-yielding cold formed steels is generally below the yield strength. This assumes that the proportional limit is 70 per cent or more of the yield strength. Most cold formed steels meet this criterion.

The basic design stress (WSD) in the Standard is 1/1.60 or 62.5 per cent of the yield strength. Thus, if the proportional limit is 70 per cent of the yield or greater, a safety margin against permanent deformation under the basic design stress is assured. Local residual stresses, if present within a tension member, will cause some portions of the cross section to reach yield strength before other portions. However as loading is increased, the stress on the yielded portions is maintained at the yield strength level.

The stress on the remaining portions eventually equals the yield strength. Thus residual stresses do not cause a reduction in load carrying capacity in a tension member.

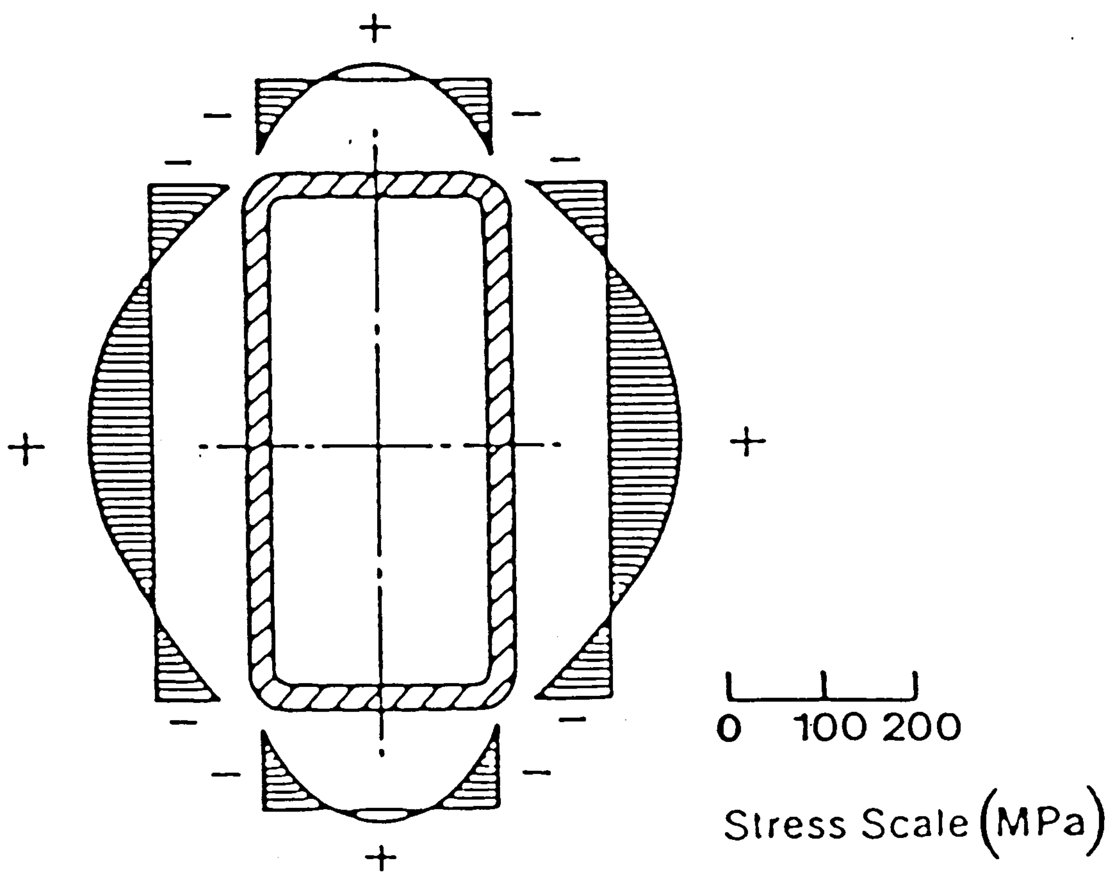
Design in Compression. In Figure 27, the test results from a tensile or compressive coupon test are compared to a full section (or stub column) compressive test. It can be seen that the proportional limit of the stub column can lie considerably below the proportional limit obtained from a coupon test. This difference is largely due to the presence of residual stresses. In hot rolled steel, residual stresses are caused by uneven cooling, while in cold formed steel they are almost entirely due to the cold working of the steel during the forming process. A representative longitudinal residual stress pattern for an as-rolled cold formed hollow structural section is shown in Figure 28. The stresses are both tensile and compressive, but the new longitudinal residual stress on the section is zero since it is in equilibrium. When the full section is tested (as in a stub column test), the stress-strain curve is linear until parts of the cross section begin to yield in areas of the section which have the maximum compressive residual stress. The maximum compressive residual stress level is, therefore, approximately the difference between the yield stress in tension and the proportional limit.

Figure 27



For members which fail by buckling, the load carrying capacity is dependent upon the slope of the stress-strain curve at the appropriate stress level. As shown in Figure 27, the slope (E_t) of the stress-strain curve above the proportional limit for a full section becomes considerably smaller than the elastic modulus (E). This reduced modulus must be taken into account in order to obtain the proper failure load. Clause 14

Figure 28



does so by assuming that the steel sections have a stub column proportional limit equal to 50 per cent or greater of the tensile coupon yield strength. This approach is in keeping with the recommendations of the Structural Stability Research Council[25]. Tests on full size columns, as well as practical experience gained over the years, indicate that the 50 per cent of tensile yield strength provision is adequate for the majority of design cases; it provides for the weakening effects of residual stress as well as other factors such as initial out-of-straightness and unintentional end eccentricity. In isolated instances, such as cold formed non-stress relieved hollow structural sections, the residual stresses can be greater than 50 per cent of the tensile yield strength.

14.4 General Design Considerations

14.4.1 Cold Work of Forming

The increase in yield strength due to cold forming steel depends largely on the degree of cold work and the difference between ultimate strength and yield strength of the virgin material. A great deal of work on identifying and quantifying these effects has been done at Cornell University.[143,144,145,146] In cold formed sections, the material in the corners is normally work-hardened to a much greater degree than in the flats. The corner effects can also be measured and predicted much more reliably than those in the flats and the latter have been essentially neglected for design purposes.

The development of an analysis to predict the increase in corner yield strength due to cold working is given in Reference 144. Its accuracy was confirmed by more than 100 tests conducted on corner samples.

This increase in corner yield strength was a function of two primary factors, the F_u/F_y ratio and the r/t ratio. The former is a measure of the potential of virgin steel for strain hardening, and the latter a measure of the degree of cold work in a corner - the smaller the r/t ratio, the greater the level of cold work and consequent increase in corner yield strength. The formula is

$$\frac{F_{yc}}{F_y} = \frac{B_c}{(r/t)^m}$$

where $B_c = 3.690\left(\frac{F_u}{F_y}\right) - 0.819\left(\frac{F_u}{F_y}\right)^2 - 1.790$

$m = 0.192\frac{F_u}{F_y} - 0.068$

F_{yc} = corner yield strength

F_y = virgin ultimate strength

r = inside bend radius

t = sheet thickness

Utilizing a weighted average calculation to determine the average full section yield of applicable members gives:

$$F_{ya} = \frac{A_c}{A} F_{yc} + \left(1 - \frac{A_c}{A}\right) F_{yf}$$

More recent research[147] used an elementary analysis assuming linear rather than power-law strain hardening and developed a simpler formula based on the hardening margin $(F_u - F_y)$ and a hardening constant determined from test results.

The new theory simply replaces the virgin yield strength by the virgin ultimate strength over a length of 5t in each 90-degree corner.

If, in the equation for F_{ya}, F_{ya} is replaced by F'_y, and F_{yf} is taken as F_y then:

$$F'_y = F_y + \frac{A_c}{A}(F_{yc} - F_y)$$

It is also assumed that in each 90-degree corner the yield strength equals F_u over a length 5t thus the apparent increase in yield strength in that length equals $(F_u - F_y)$. Therefore,

$$F'_y = F_y + \frac{5D}{W^*}(F_u - F_y)$$

where D = number of 90 degree corners. If other angles are used, D is the sum of the bend angles divided by 90 degrees

W* = ratio of the length of the centerline of the full flange of flexural members, or of the entire section of tension or compression members, to the thickness

The ratio of bend radius to thickness, r/t, might be expected to be a significant parameter. However, when r/t is small the volume undergoing strain hardening is also small and although the increase in strain, and therefore the yield strength is large. The converse is also true resulting in a net effect which is essentially independent of r/t. The increase in yield strength is also proportional to the bend angle. The test results show that the total increase in yield strength is proportional to the stress margin (F_u - F_y) acting over a length of approximately 5t of the plate at the corner.

The F_y' formula is much simpler to use than F_{ya} and for full section properties provides good agreement with the theory developed in Reference 147. The F_y' formula, although increasing yield strength over an empirically determined area of $5t^2$ for a 90-degree corner regardless of radius, has a similar net effect as F_{ya} since in practice a tighter radius represents a greater increase in yield strength.

14.4.2 Maximum Allowable Flat Width Ratios for Compression Elements

This clause contains limitations on permissible flat-width ratios of compression flanges and of webs of beams. As for all limitations, the exact values indicated in these sections are to some extent arbitrary. They do, however, reflect a body of experience and are intended to limit practical ranges.

The use of simple lips as edge stiffeners is restricted to elements with w/t not exceeding 60.

For w/t = 60 the minimum required d_ℓ/t is 10.9 by Clause 14.4.5.2. For customary corner radii, this results in a w/t of the lip of about 8 or 9. Hence if compression elements with w/t significantly exceeding 60 were stiffened by simple lips, the w/t for the lip would exceed $165/\sqrt{F_y}$ necessitating a reduction in the allowable compressive stress.

The limitation to w/t = 90 for flanges with edge stiffeners other than lips reflects the fact that thinner flanges are liable to be damaged in transport, handling and erection.

Much the same can be said for the limitation to w/t = 500 of web-stiffened compression elements. The Note specifically states that wider flanges are not unsafe but that stiffened flanges exceeding w/t = 250 and unstiffened flanges exceeding w/t = 30 are likely to develop noticeable, though structurally harmless, distortions at design loads. In both cases the upper limit is set at twice that ratio at which first noticeable deformations are likely to appear, based on observation of such members under test.

It should be noted that for unstiffened elements the allowable stress decreases very rapidly with increasing w/t ratios beyond w/t = $165/\sqrt{F_y}$. Consequently, in designing shapes for load carrying purposes, the use of unstiffened elements with w/t substantially exceeding $165/\sqrt{F_y}$, will usually be found entirely uneconomical. Design stresses for w/t ratios up to 60 are provided in Clause 14 as cold formed shapes are often dictated by other than structural considerations.

14.4.3 Maximum Allowable Web Depths

The limit $h/t < 200$ applies to webs typical in cold formed construction. Such webs are generally unstiffened and are connected to the flanges through rounded corners; causing loads to be introduced into the web with some eccentricity.

For those situations where adequate stiffeners or appropriate framing details transmit concentrated loads or reactions into the web, the maximum h/t when bearing stiffeners only are used has been extended to 260 and when both bearing and intermediate stiffeners are used to 300. Stiffeners or, in the case of reactions, appropriate framing details must be provided for transmission of reaction without causing web distortion.

14.4.4 Properties of Sections

14.4.1.1 General

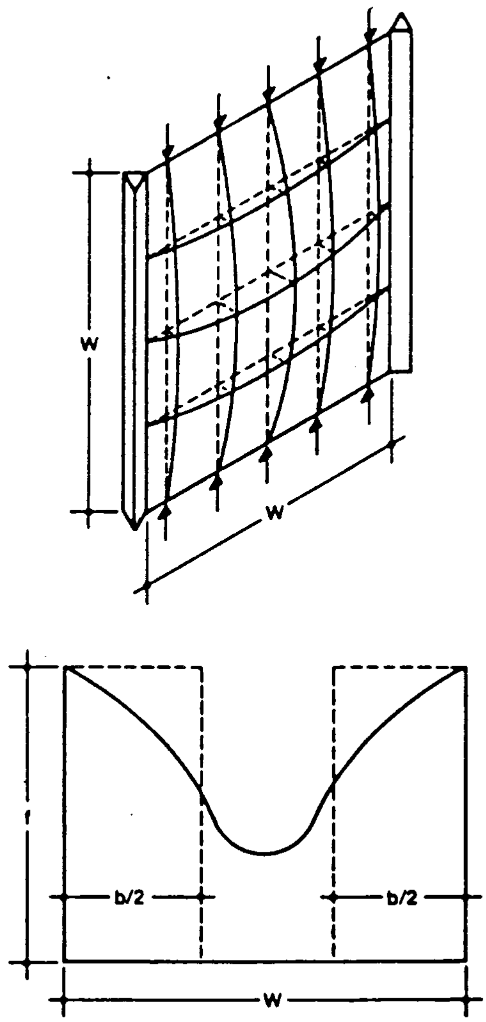
In order to take into consideration the post-buckling strength, the full section properties of a member (i.e. area, moment of inertia, section modulus, excepting radius of gyration) are transformed into effective properties. Depending on the edge conditions, the elements composing the cross section of the member are classified into stiffened and unstiffened elements as defined in Clause 14.2.

For compression elements, effective widths determined in accordance with Clause 14.4.4.2 are used in the calculation of the effective properties.

14.4.4.2 Effective Width

The effective width design concept for a stiffened compression element in which the two edges parallel to the compression force are supported can best be explained by the grid model developed in Reference 148. As shown in Figure 29 the plate element is assumed to be composed of a series of columns connected by ties in the transverse direction. As the buckling strength of the plate is reached, the lateral deflection of the columns in the middle of the plate induces membrane stresses which are represented by the tension developed in the transverse ties. These ties restrain the increase in deflection and thus prevent this column from buckling. With further increase in load, this column remains stable but it is unable to carry an increased load. However, the neighbouring columns can handle the increase. Therefore, the stress in the plate is redistributed to allow the compression to be concentrated near the edges where restraint is most effective. Thus the ultimate load of the stiffened plate is governed not by the initial buckling but by the yielding of the edges. The maximum compressive resistance developed by such a plate is considered to be equivalent to that of a flat plate with an effective width b over which the stress is uniform and equal to the actual stresses at the edge of the plate.

Figure 29



14.4.4.2.1 Compression Elements and Sub-elements of Multiple-Stiffened Elements

An effective width concept for the design of thin plates in compression was introduced in 1932.[149] The theory equated the critical stress on the total width to the yield stress on the effective width resulting in:

$$\frac{b}{t} = 1.9t \sqrt{\frac{E}{F_y}}$$

Based on a series of tests on light gauge steel beams at Cornell University[150,151,152,153] the above equation was modified to be:

$$\frac{b}{t} = 1.9 \sqrt{\frac{E}{f_{max}}} \left[1 - \frac{C}{(w/t)} \sqrt{\frac{E}{f_{max}}} \right]$$

where f_{max} = stress at edges of stiffened compression element

C = constant

The constant C was originally taken to be 0.475 but based on a re-study of the original tests and on additional test results, the somewhat less conservative value of C = 0.415 was introduced.

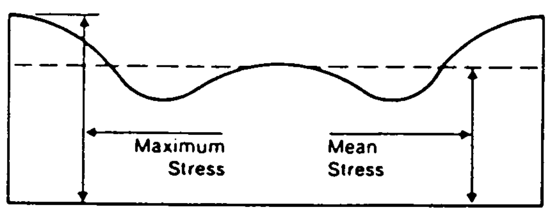
By substituting E = 203 000, f_{max} = f, the actual stress in the compression element and introducing k = 4.0 for stiffened compression element the effective width thickness ratio becomes

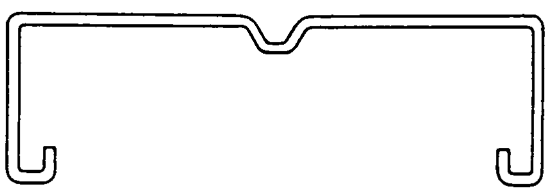
$$\frac{b}{t} = 428 \sqrt{\frac{k}{f}} \left[1 - \frac{94}{(w/t)} \sqrt{\frac{k}{f}} \right]$$

By introducing the plate buckling coefficient, k, this expression can be applied to other edge conditions. For example for an unstiffened compression element a value of k = 0.5 is used in this clause.

The formula in Clause 14.4.4.2.1 also calls for a further reduction in the effective width ratio b/t by a factor R when w/t > 60 for a sub-element of a multiple-stiffened compression element and when w/t > 760 for a stiffened compression element without intermediate stiffeners and with only one edge connected to a web. The need for reduction in the effective width ratio in these cases is attributed to "Shear Lag" in the flange of a flexural member and to the change in geometry due to the tendency of that portion of flange remote from the webs to deflect towards the neutral axis. The resulting stress distribution in the flange is shown in Figure 30. It should be noted that this reduction in efficiency does not substantially detract from the very considerable gain in structural economy obtained by intermediate stiffeners.

Figure 30





14.4.4.2.2. Effective Area of Stiffeners

It is necessary to reduce areas of edge stiffeners and intermediate stiffeners for section property determination of stiffened compression elements for which w/t exceeds 60 because of the effect of "shear lag".

14.4.4.2.3 Unusually Short Spans Supporting Concentrated Loads

In beams with unusually wide flanges (relative to their length), the distribution of stress across the width of the flange is non-uniform. This is due to the phenomenon known as "Shear Lag". The effect is similar to that in the stiffened compression elements though for entirely different reasons. As before, the simplest way of accounting for this stress variation in design is to replace the non-uniformly stressed flange of actual width w' by one of reduced, effective width subject to uniform stress. The provisions in this clause are based on the analysis and supporting experimental evidence in Reference 154. It should be noted that the use of a reduced width for stable wide flanges is required only for concentrated load. For uniform load, the width reduction due to shear lag for any but unrealistically large width-span ratios is so small as to be practically negligible.

The phenomenon of shear lag is of considerable consequence in naval architecture and aircraft design. In cold-formed members it is infrequent that the flanges are so wide as to require significant reduction according to the provisions of this clause.

14.4.5 Stiffeners for Compression Elements

14.4.5.1 General

As seen in Figure 29, the post-buckling strength of a compression element is enhanced when its edges are supported or stiffened in such a manner that out-of-plane distortion is prevented. Usually this is achieved by connecting webs to flanges as in hat or box sections. In other cases, only one longitudinal edge is stiffened by a web while support of the other is provided by an edge stiffener which may take the form of a simple lip, such as in lipped channels. Sometimes compressive elements with large flat-width ratios are required for reasons other than structural considerations. In such cases, the addition of "intermediate" stiffeners between webs or between a web and an edge stiffener can improve the structural efficiency.

Clauses 14.4.5.2 and 14.4.5.3 provide the minimum rigidity requirements for edge stiffeners and intermediate stiffeners.

14.4.5.2 Edge Stiffeners

Originally the formulas for edge stiffeners were established by an elastic buckling analysis to give a simple but a close fit to make the buckling stress of an edge-stiffened flange equal to that of an identical flange stiffened by webs along both edges. The analysis was restricted to the elastic range, and no attempt was made to include the post buckling strength which is difficult to analyse. Although the formula is an approximation, it has been found in practice to provide adequate stiffness. The equation includes a term that reflects a theoretical dependence on the yield strength of the material. This dependence is slight and it rarely exceeds the effect of manufacturing tolerance. In view of the approximations involved, this dependence of the yield strength is without significance and has been eliminated.

To support adequately the edge of a stiffened compression element the stiffener must provide an adequate moment of inertia about its centroidal axis. When $w/t < 290\sqrt{\frac{k}{f}}$ a stiffened compression element can buckle at the yield stress therefore the moment of inertia that is required is at least $9t^4$. When $w/t > 290\sqrt{\frac{k}{t}}$ the moment of inertia required increases with increasing w/t ratios of the stiffened compression element resulting in a more rigid stiffener. By setting $9t^4$ equal to $(d_\ell)^3 t/12$ and solving for d_ℓ the minimum depth for simple lips is obtained.

14.4.5.3 Intermediate Stiffeners for Multiple-Stiffened Elements

Following the reasoning presented in the Commentary on Clause 14.4.5.2 it would be reasonable to assume that the minimum rigidity of an intermediate stiffener is twice that of an edge stiffener because each intermediate stiffener must support two adjacent sub-elements.

Clause 14.4.5.3(a) and (b) reflect the effect of shear lag on multiple-stiffened flanges having compression sub-elements with an effective width-thickness ratio less than the flat-width ratio. The only intermediate stiffeners that are considered effective are those adjacent to full webs as additional stiffeners would have two or more sub-elements between themselves and the nearest shear-transmitting element.

When intermediate stiffeners are spaced sufficiently close the compression sub-elements have no tendency to buckle individually. The entire assembly of sub-elements and intermediate stiffeners between webs behaves like a single compression element, therefore, the entire element is assumed to have an equivalent thickness t_e, as given in Clause 14.4.5.3(c). Naturally, the real thickness, t, must be used to calculate the properties of the cross-section of which this element is a part.

14.5 Member Resistance

14.5.1 General

This clause outlines the resistance factors (LSD) and factors of safety (WSD) to be used in Clause 14 only. Where buckling is a consideration, as for columns and laterally unsupported beams, the resistance factor is actually 0.75. In the expressions for resistance this value is obtained as 0.833ϕ = $0.833(0.90) = 0.75$.

14.5.2 Axial Tension

See commentary on Clause 13.2.

14.5.3 - Flexural

14.5.3.2 - Single-Web Members (I, Z, or Channel Shaped Members)

The equations given in this clause include the contribution of the St. Venant torsion stiffness to lateral buckling of beams which may sometimes increase the buckling stress by 15 to 18 per cent, for example in channel sections.

The buckling stress of I-beams, F_{cr}, is given by:[148]

$$F_{cr} = \frac{\pi^2 Ed}{2\omega S_{xc} L^2} \left(I_{yc} - I_{yt} + I_y \sqrt{1 + \frac{4GJL^2}{\pi^2 I_y Ed^2}} \right)$$

Assuming that $\frac{4GJL^2}{\pi^2 I_y Ed^2} \ll 1.0$

F_{cr} may be re-written by a Taylor's series expansion of the square root as:

$$F_{cr} = \frac{\pi^2 EdI_{yc}}{\omega S_{xc} L^2} + \frac{G\ J}{\omega dS_{xc}}$$

$$F_c = F_{be} + F_t$$

where $$F_{be} = \frac{\pi^2 EdI_{yc}}{\omega L^2 S_{xc}}$$

$$F_t = \frac{0.335GAt^2}{\omega dS_{xc}}$$

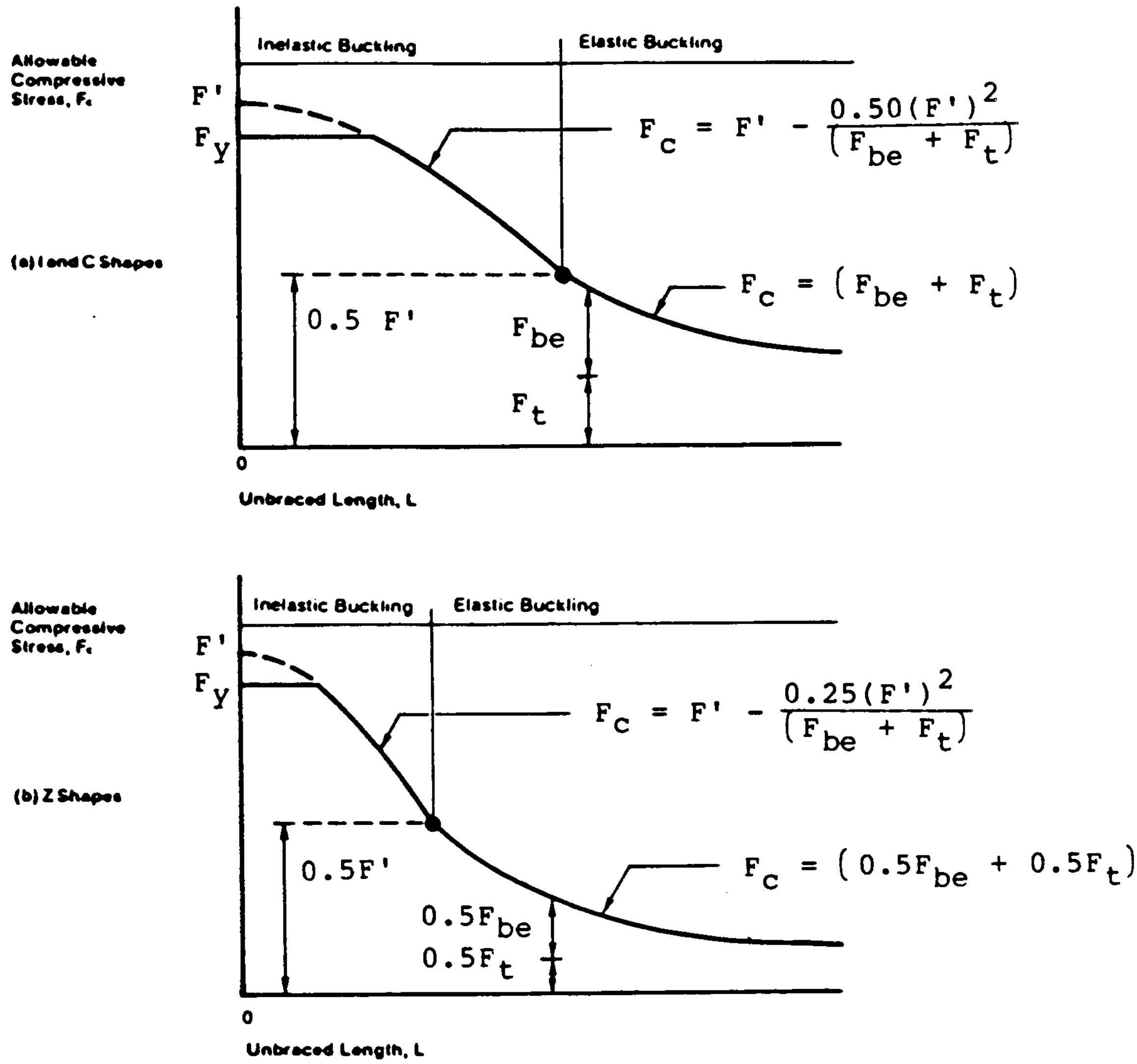
The equation for F_{cr} is only valid when f_{cr} is below the proportional limit. The allowable buckling stress in the inelastic range is approximated by a parabola[155] between the basic design stress F_y (at zero unbraced length) and the curve given by $F_c = F_{be} + F_t$.

Figure 31 depicts the requirements of Clause 14.5.3.2.

14.5.3.3 Closed Box Flexural Members

While the closed box section has superior torsional stiffness, the possibility of lateral buckling of such members is not ruled out completely. Neglecting the warping stiffness, the elastic buckling stress (in the pure bending case) is given by:[155]

Figure 31



$$F_{cr} = \frac{\pi}{LS_{xc}} \sqrt{GJEI_y} = \frac{\pi E}{LS_{xc}} (G/E)^{1/2}(JI_y)^{1/2}$$

Substituting for G/E = 78 000/203 000, the allowable buckling stress is computed as:

$$F_c \approx 1.95 \frac{E\sqrt{JI_y}}{LS_{xc}}$$

This stress may be applicable to long beams. For shorter beams the stress is governed by yielding.

14.5.3.4 - Bending in Webs

It is well known that web elements in pure bending do not fail at the theoretical buckling stress but develop post-buckling strength. Recent research into the action of beam webs in bending[156,157,158,159,160] has shown that the post-buckling strength of webs varies with the depth to thickness ratio, the bending stress ratio, the type of compression flange and the yield point of the material. Based on this research a number of different design methods have been developed. The design technique used in Clause 14 is the post-buckling method[156] which provides the most straight forward synthesis of the factors involved.

The post-buckling equation includes a post-buckling factor Φ, which modifies the basic limiting stress to account for the post-buckling behaviour of the web. This method also recognizes the effect of the flange/web interaction on the web bending capacity. For very wide flanges early buckling may occur under load and then propagate into a premature buckling of the web.[156] The definitions for "web" and "flange" of sections in bending clearly define the point at which the web ends and the flange begins. The limiting value of bending stress for the web, F_{bw}

occurs at this point labelled "A" in Figure 32. It is not permissible to use the effect of cold work when applying Clause 14.5.3.4. Therefore, F_{bw} in the web cannot exceed F_y in the flange which causes a discontinuity of stress at A. The compressive stress, F_{bw} in the web, is then the lesser of F_y or F'_y in the flange and F_{bw} in the web.

Webs in bending do not fail at the theoretical buckling stress, but develop sizeable post-buckling strength.[161] For this reason only a small factor of safety, 1.23 is used against buckling. The curve describing the allowable bending stress is illustrated in Figure 33.

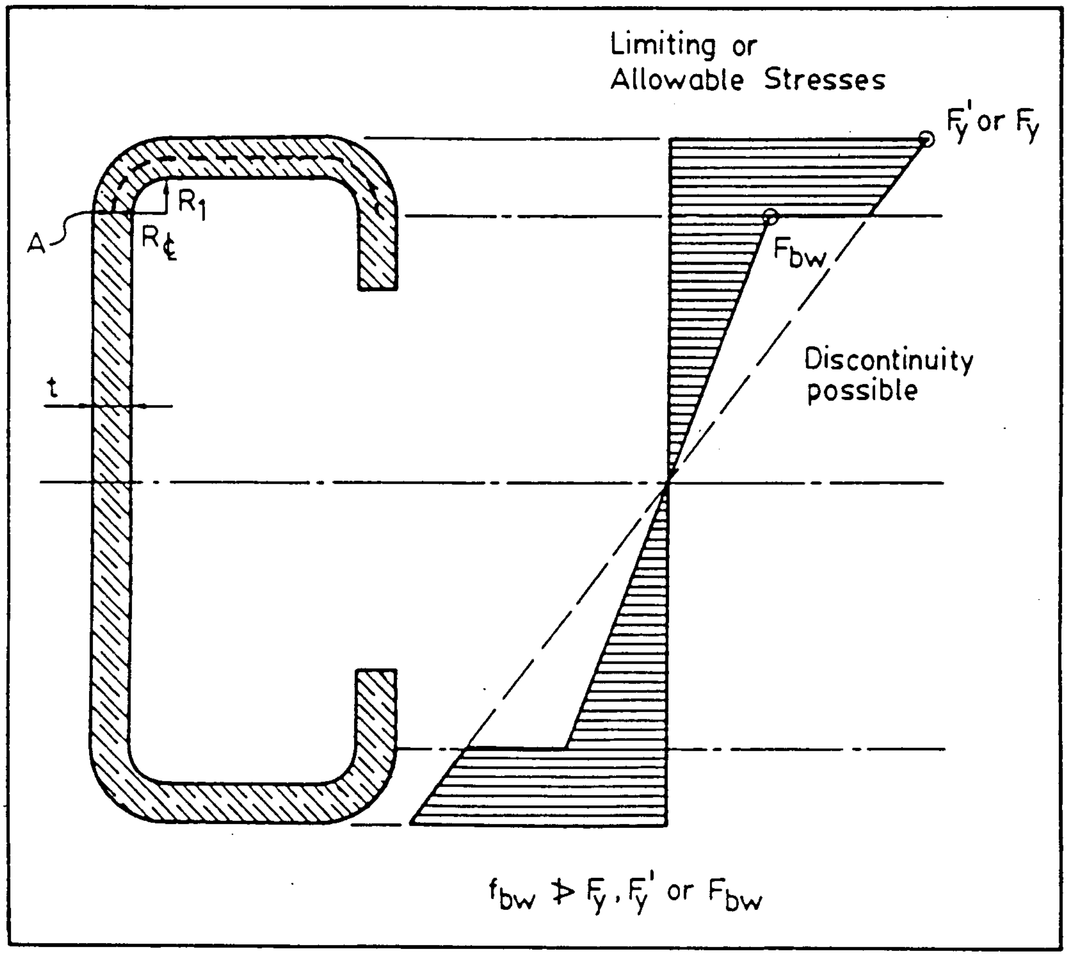
14.5.3.5 Shear in Webs

Shear stress expressions are provided for webs with and without stiffeners. These provisions are based on the results of a study of beam webs loaded primarily by shear stress.[162]

The shear capacity of webs is dependent on the depth to thickness ratio, the support conditions along the edges, the aspect ratio of the web element and the material properties. The shear strength is governed by yielding when h/t is small, by elastic buckling when h/t is large and by inelastic buckling when h/t is in the intermediate range. The elastic critical shear stress is given by:

$$\tau_{cr} = \frac{k_v \pi^2 E}{12(1 - \mu^2)(h/t)^2}$$

Figure 32

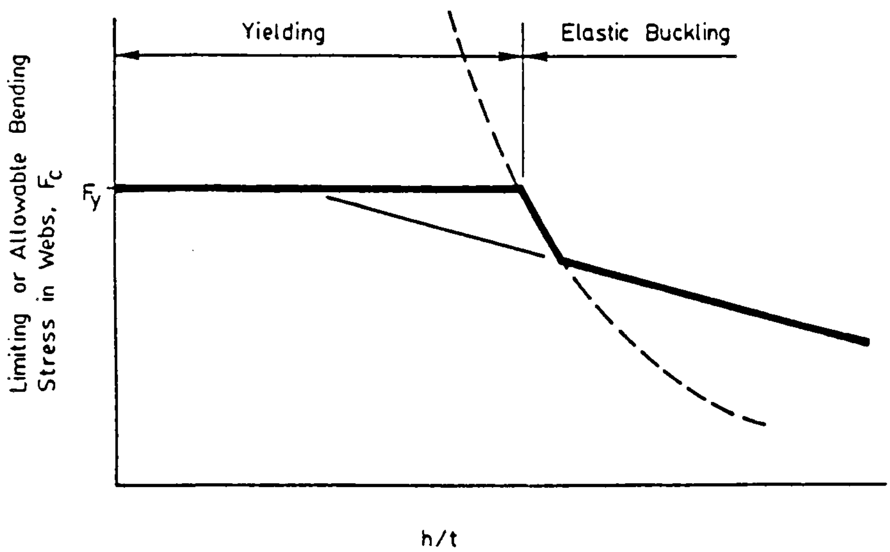


Where h/t is the intermediate range, the shear capacity of the web element is governed by inelastic buckling.[163]

Clause 14 allows the designer to use a larger shear capacity for beam webs with adequate transverse stiffeners.

In WSD Clause 14.5.3.5 a safety factor of 1.44 is used to prevent shear yielding. The reduced factor of safety for yielding is justified by the minor consequences of such yielding as compared to flange yielding. For inelastic and elastic buckling, the safety factors are 1.67 and 1.71 respectively.

Figure 33



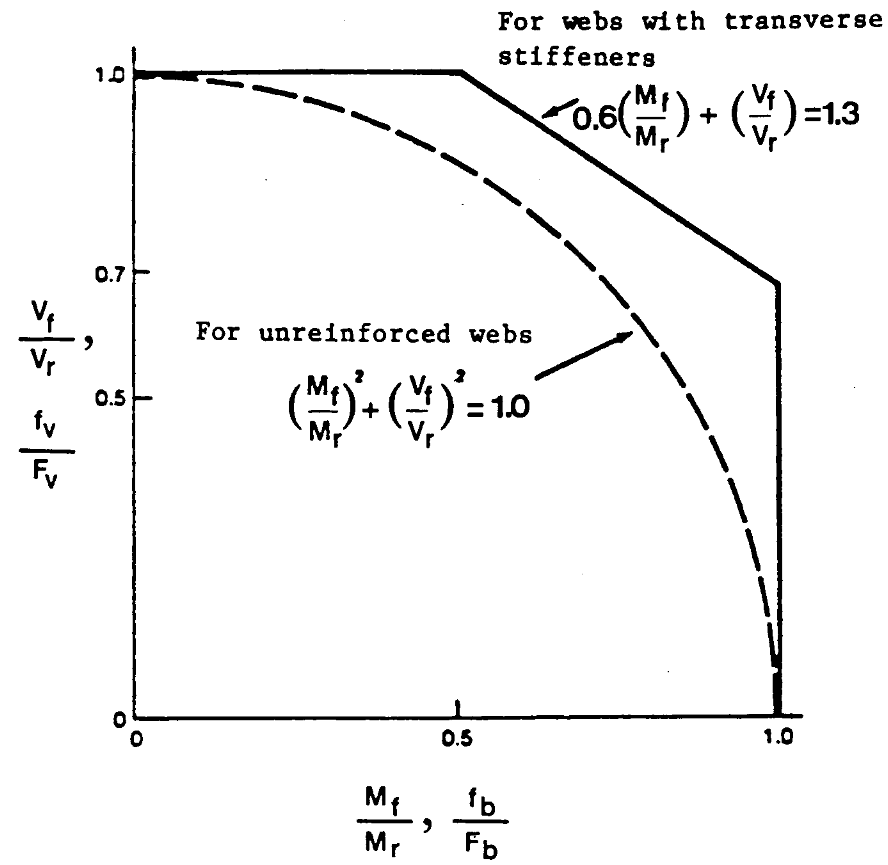
14.5.3.6 Combined Bending and Shear in Webs

This clause provides for the interaction between bending and shear in webs and their effect on the capacity of the web element. There are two interaction equations one for unreinforced flat webs and one for beam webs with adequate transverse stiffeners. These two expressions are shown graphically in Figure 34.[164]

14.5.3.7 - Web Crippling

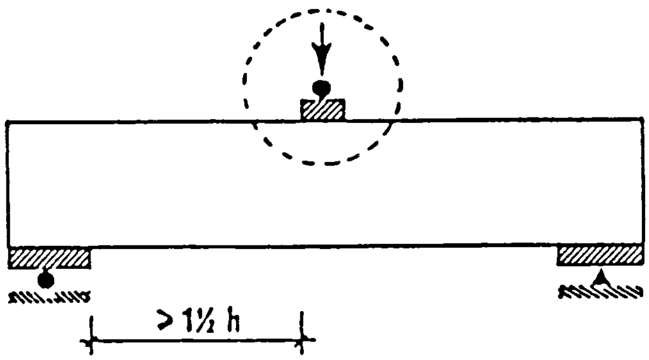
When concentrated loads or reactions are applied to cold formed steel members with unreinforced webs, crippling of the web may result.

Figure 34

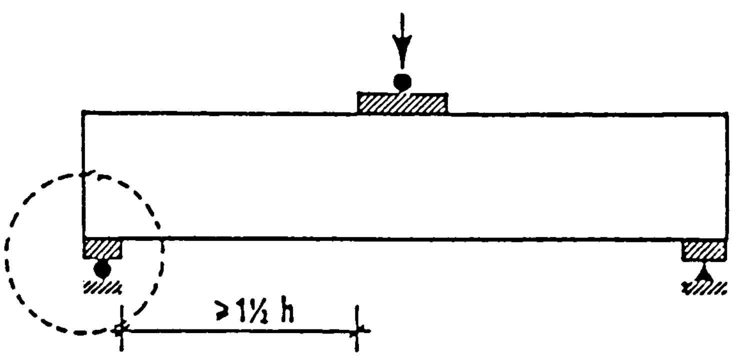


This clause provides equations to calculate the web crippling resistance of I-Beams, shapes having single webs and hat and deck sections. Equations are presented for both single and opposite flange loading for both end and interior locations (Figure 35). These equations are based exclusively on experimental evidence[165,166] as a theoretical analysis is extremely complex.

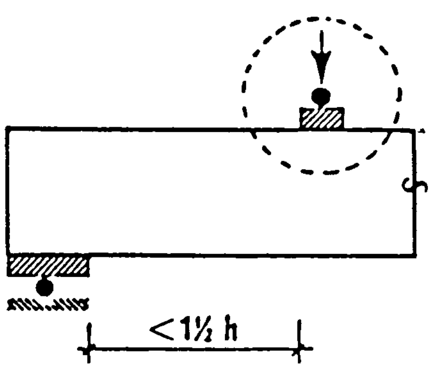
Figure 35



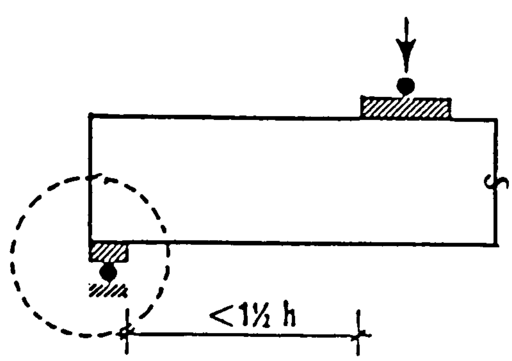
(a) Interior Single Flange Loading



(b) Exterior Single Flange Loading



(c) Interior Opposite Flange Loading



(d) Exterior Opposite Flange Loading

Research conducted in the mid to late 1970's[165,166] has indicated that the web crippling capacity varies with the loading conditions and section type.

The equations in Tables 7 and 8 of the Standard are based on research done at the University of Missouri-Rolla[165] with the following modification. The bearing length term in the equation for interior opposite flange loading for shapes having single webs was changed from (1.0 + 0.0013N) to (1.0 + 0.01N) which is more in line with the factors used in the other equations for shapes having single webs as well as the results of other research[166]. The equations presented are only applicable to sections with vertical webs.

14.5.3.8 Combined Web Crippling and Bending

Several researchers[165,166,167,168] have investigated cold formed sections subjected to web crippling and bending and have all found that there is an interaction between the two loading conditions. The individual researchers all recommend similar but slightly different interaction equations and the equation given in Clause 14 is based on a consensus of the research findings.

14.5.4 Transverse Stiffeners for Beam Webs

14.5.4.2 Intermediate Stiffeners

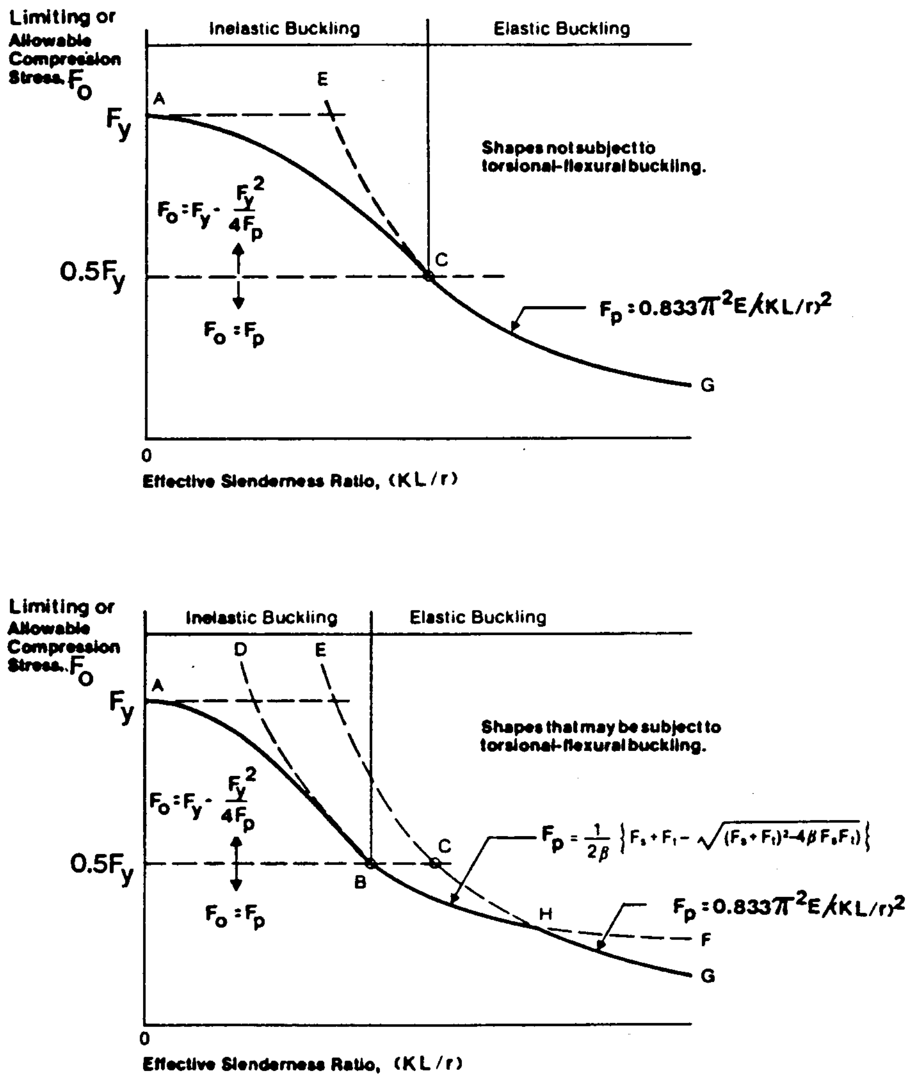
The requirements for intermediate stiffeners were adopted from Reference 170. The equations for determining the minimum required moment of inertia and the minimum required gross area of attached intermediate stiffeners are based on the studies summarized in Reference 171. In this reference, test data show that even though the shear resistance formulas of section 14.5.3.5 are based on the buckling strength of web elements, rather than on tension field action, it is still necessary to provide the required moment of inertia and gross area of intermediate stiffeners. This is because the flanges of cold-formed steel beams often are quite flexible, as compared with the flanges of hot-rolled shapes and plate girders. The minimum value of (h/50) was selected from Reference 156.

No tests on the rolled-in transverse stiffeners have been made in the experimental program reported in 171. Therefore, lacking reliable information it is required that the dimensions and load resistance of rolled-in stiffeners be determined by tests.

14.5.5 Axially Loaded Compression Members

The failure of an axially loaded compression member may be due to yielding, local buckling, flexural buckling (elastic or inelastic), torsional buckling or torsional-flexural buckling, for members whose shear centre and centroid do not coincide (singly-symmetric sections). This is depicted in Figure 36.

Figure 36



Very short columns fail by yielding if local buckling is precluded by combined yielding and local buckling.

Theoretically, longer members with point symmetry may fail by pure torsional buckling, if the torsional stiffness is sufficiently small in comparison with the flexural stiffness (e.g., cruciform cross sections). This mode of failure is, however, not usual for the shapes occurring in cold formed structural practice, and Clause 14 contains no provisions against pure torsional buckling.

Very slender columns buckle elastically, either in pure flexure or in torsion combined with flexure. Torsional-flexural buckling is precluded either:

(a) if the cross section has point symmetry, because shear centre and centroid coincide; or

(b) if the torsional stiffness is large as in closed cross sections.

Short and intermediate columns fail by inelastic buckling, either in flexure or torsion combined with flexure. This occurs for columns for which the calculated value of F_p is greater than $0.5F_y$, approximately, as represented by curve segments AB or AC in Figure 36. These curves are Johnson parabolas, effecting a gradual transition between A, and B or C where they are tangent to the elastic buckling curves. Since the torsional-flexural stress does not depend solely on the effective slenderness ratio, the curve ABF in Figure 36 is only schematic. The location of point B and intersection point H will depend on the cross section.

For all columns the stress F_a is thus expressed as a function only of two indices F_y and F_p.

For sections that may be subject to torsional-flexural buckling, it may be cumbersome to compute the warping constant, C_w of the cross section. The following procedure may sometimes avoid the computation of C_w.

As a first trial, one can neglect the warping contribution to torsional resistance and calculate

$$F_t = \frac{0.833CJ}{Ar_o^2}$$

Now if $F_p = \frac{1}{2\beta}\left[F_s + F_t - \sqrt{(F_s+F_t)^2 - 4\beta F_s F_t}\,\right] > F_p$ as given by Clause 14.5.5.2, flexural buckling is critical and no further computations are necessary. On the other hand, if this isn't the case, one may still neglect the warping contribution to torsional resistance, which yields a conservative stress F_a.

14.5.5.6 Built-up Members

A built-up member subjected to lateral forces deforms by a combination of overall bending, related to the rigidity EI, and local inter-fastener deformations, related to the shear rigidity.

The resistance of a member to torsion is derived from the St. Venant torsional stiffness, GJ, and, from the warping rigidity, E(h/t).

All of the above four properties of a member may need to be considered, either separately or in combination, when determining the critical force for an axially loaded compression member.

Built-up members may fail by flexural or torsional buckling, or a combination of both. Local buckling will rarely occur.

The critical stress for the flexural or torsional buckling mode represents the maximum possible capacity. There is no post-buckling reserve.

Ideally the stress to cause overall flexural buckling under the action of an axial compressive force is given by

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Where two or more angles are connected together, the inter-fastener deformations will reduce the critical load. This influence is related to the slenderness:

$$\left(\frac{L}{r_z}\right) = \frac{a}{r_z}$$

in which

a = distance between fasteners

r_z = appropriate radius of gyration of the single element about an axis parallel to the composite axis

The critical stress is then given by:

$$F_{cr} = \frac{\pi^2 E}{(KL/r)^2 + (a/r_z)^2}$$

14.5.6.1 Axial Compression and Bending

See Commentary on Clause 13.7.

14.5.7 Single Angles Loaded Through One Leg

If an equal leg angle is connected to a supporting member by a fastener at the centre of one leg, the member is subjected to axial load combined with a moment about the axis perpendicular to the Z-axis.

Because there is no warping rigidity the critical stress for elastic torsional buckling is:

$$F_t = GJ/I_p$$

where

I_p = polar moment of inertia about the shear center (at the heel for plain angles)

This stress can be related to the buckling formula for columns by equating it to the Euler stress. This yields an equivalent slenderness.

$$(L/r) = \pi \sqrt{\frac{EI_p}{GJ}}$$

For equal leg angles this becomes

$$(L/r) = 5b/t$$

where b = leg width measured from the intersection of the median lines of the legs

t = thickness

For unequal angles the longer leg is used, giving conservative predictions.

Failure is by lateral torsional buckling at a stress given closely by

$$F = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2 + \left(\frac{5b}{t}\right)^2}$$

14.5.8 Wall Studs

Cold formed steel studs, used in load bearing walls or partitions, are faced on both sides with a variety of wall materials such as gypsum board, fiberboard, etc. Although the main function of this wall sheathing is to form the outer and inner wall surfaces, they also provide bracing for the wall studs.

The sheathing material provides shear rigidity which limits the lateral deflections and flexural rigidity which limits the rotations. The stud to wallboard connections also provide rotational restraint.

General procedures for determining the strength of studs braced on one or two flanges are given in References 172 and 173. A computer program has also been developed.[174]

The strength provisions of Clauses 14.5.8.1.1 and 14.5.8.1.4 consider the cases where identical and dissimilar sheathings are applied to the flanges of the studs. In Clause 14.5.8.1 only the restraint due to the shear rigidity of the sheathing is considered.

The provisions in the specification are conservative to account for the situation where the ends of the wall studs are not free to rotate about both axes perpendicular to the stud axis.

14.5.8.1 Studs in Compression

The provisions of this clause insure that (a) column buckling in the plane of the wall between fasteners, including when one fastener is missing or ineffective, is prevented (b) overall column buckling is prevented and (c) the shear rigidity provided by the wallboard has sufficient distortion capacity to prevent excessive lateral bending. The shear rigidity equation and the sheathing parameters given in Table 10 are based on small scale tests[172,173]. The values $\bar{q}_o$ and $\bar{\gamma}$ were determined at 0.8 times the ultimate shear load of the test assembly described in this reference. To determine the parameters for other types of materials consult References 172 and 173 for the necessary procedures.

In provision (c) the formulas are complex therefore a value of the limit stress is assumed, the shear strain using the limit stress is calculated and then this value is checked against the limit shear strain of the sheathing material. Although this procedure is one of successive approximations if the least F_o value from (a) or (b) is tried and is satisfactory no iteration is necessary.

14.5.8.1.4 When situations exist that do not meet the requirements of Clause 14.5.8.1.1 accepted published analytical methods[172,173] may be applied. For the case of dissimilar sheathing on each side of the stud this clause allows the use of 14.5.8.1.1 if the least shear rigidity and the least limit shear strain values are used for both wallboards.

14.5.8.2 - Combined Axial and Bending in Studs

This section gives provisions formulated on the basis of precedent for other types of members and engineering judgement, and believed to be sufficiently accurate.

14.6 - Connections

14.6.1 - Fastening Devices

A considerable variety of means of connection finds application in cold formed construction. This is particularly true for the thinner elements. Without any claim for completeness, these may be listed as follows:

(a) Welds, which may be sub-divided into resistance welds, and arc welds;

(b) Bolts;

(c) Rivets, while hot rivets have little application in cold formed construction, cold rivets find considerable use, particularly in special forms, such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, explosive rivets, and others. Most of these are proprietary products;

(d) Screws, mostly self-tapping screws of a considerable variety of types;

(e) Special devices, among which may be mentioned:

   (i) Metal stitching, achieved by tools which are special developments of the common office stapler;

and

(ii) Connecting by upsetting, by means of special clinching tools which draw the sheets into interlocking projections.

The Standard contains design data only for welded and for bolted connections. Classes (c), (d), and (e), above, mostly refer to a variety of proprietary devices in regard to which information on strength of connections must be obtained from manufacturers or from tests carried out by or for the prospective user.

14.6.2 Welded Connections

14.6.2.1 Arc Welds

Fusion welding is used for connecting cold formed steel members to each other as well as connecting such members to heavy, hot rolled, steel framing (such as floor beams, or girders of the steel frame). It is used in fillet welds, butt welds (rarely), and plug welds.

The provisions of this clause for a thickness greater than 3.5 mm are based on an experimental test program at Cornell University.[175] Data were collected on tests of connections made in steel fabricating shops, made under field conditions, and fabricated in the Cornell Laboratory under simulated field conditions.

Rupture rather than yielding provided a more reliable criterion of failure.

The provisions for a thickness less than 3.5 mm were established from tests on sections with single and double cover plates.

14.6.2.2.3 Arc-Spot Welds (Puddle Welds)

Arc spot welds tested at Cornell University[175] failed by:

(a) simple shear failure of the weld metal in the plane of the faying surface

(b) tearing of the loaded side of the sheet (starting by tearing along the weld contour and then continuing by progressing across the sheet)

(c) tearing along the contour of the weld on the tension side and then plowing of the weld into the end material as that material buckles and shears

(d) a combination of (b) and (c).

The provisions of Clause 14.6.2.2.3 protect against the above mentioned failure modes.

Inelastic out-of-plane deformation often precedes or accompanies failure of the plate tearing type, therefore, by closely spacing the welds this behaviour is prevented.

14.6.2.2.4 Fillet Welds

The provisions of this clause are based on failure by tearing caused by applied shearing or tensile forces (longitudinal or transverse loading, respectively). Shear failure of the weld is prevented because the weld material has a higher strength than the base metal. Sections greater than 3.8 mm may have a throat less than the thickness of the cover plate and thus failure may be from a tear in the weld rather than in the plate material. The last provision accounts for this.

14.6.2.2.5 Flare Groove Welds

Flare-groove welded cold-formed channels, loaded either transversely or longitudinally fail by tearing along the contour of the weld, therefore the provisions of this clause are to prevent failure by sheet tearing. For channel sections with a thickness greater than the effective throat of the weld, weld failure may be critical. The last provision accounts for this.

14.6.2.3 Resistance Welds

Spot welding in its normal form as well as by projection welding is probably the most important means of shop connecting thinner sheets in cold formed steel fabrication. The equation gives the shear resistance (LSD) or allowable shear stress (WSD) per spot weld, depending exclusively on the thickness of the thinnest outside connected sheet. Although the equation is for uncoated sheets, in practice it is also used for galvanized sheet.

14.6.3 Connections Made by Bolts, Rivets and Screws

14.6.3.2 Bearing Resistance of Bolted Connections

The bearing resistance (LSD) or the bearing stress (WSD) is based on the tensile strength. Available test data have shown that the bearing strength of bolted connections depends on the thickness of the connected parts, whether the joint is in single or double shear, the F_u/F_y ratio, and the use of washers.[176,177] The bearing resistances (LSD) or bearing stresses (WSD) are given for a number of different cases in the Standard. Some thicknesses of material are not covered in Clause 14.6.3.2, therefore, the bearing stress must be determined by tests in accordance with Clause 14.8.

14.6.3.4 Tension Resistance of Net Section

Clause 14.6.3.4 is applicable when the thinnest connected part is less than 4.76 mm thick. For thicker materials Clause 13.10 shall apply. The allowable tension resistance (LSD) or allowable tensile stress (WSD) given for the net section of a connected member is determined by the tensile strength of the connected part, based on the type of joint[181,183] and whether washers are used.

14.6.4 Connections in Built-up Members

14.6.4.1

(a) The principal two-flanged shapes which are cold formed from a single sheet without welding are channels and zees, with or without lips. Except for light loads, I-shaped sections are often preferable for compression members. In cold formed construction these can be produced by connecting two channels back to back.

For two channels connected to function as a single compression member, it is necessary to make the longitudinal spacing between connections (e.g., spot welds) small enough to prevent the component channels from buckling individually about their own axis parallel to the web at a load smaller than that at which the entire compression member would buckle.

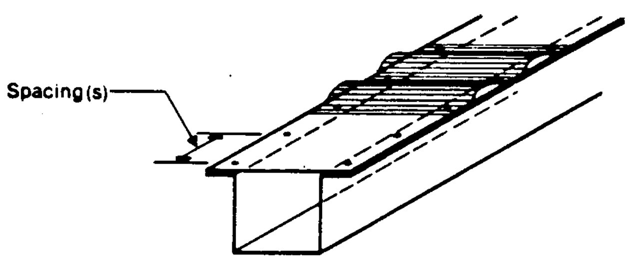
A concentrated load applied to a double channel beam tends to separate the flanges. See Commentary on Clause 14.7.3.

Compression members can also be made by connecting two channels tip-to-tip to form a box shape. Lipped channels facilitate fabrication of such shapes by welding.

14.6.5 Spacing of Connections in Compression Elements

If compression elements are joined to other parts of the cross section by intermittent connections, such as spot welds, these connections must be sufficiently closely spaced to develop the required strength of the connected element. For instance, if a hat section is converted into a box shape by spot-welding a flat plate to it, and if this member is used as a beam with the flat plate up, i.e., in compression as in Figure 37, then the welds along both lips of the hat must be spaced so as to make the flat plate act monolithically with the hat. If welds are appropriately placed, this flat plate will act as a stiffened compression element with a width w equal to the distance between rows of welds, and the section properties can be calculated accordingly. The spacing, s, must not exceed the lesser of the values determined in Clause 14.6.5(a), (b) and (c).

Figure 37



Clause 14.6.5(a) requires that the necessary shear strength be provided by the same standard structural design procedure as is used in calculating flange connections in riveted or welded plate girders or similar structures.

Clause 14.6.5(b) ensures that a part of the sheet between adjacent connections will not buckle as an Euler column at a load less than that corresponding to the overall buckling load of the member. This provision is based on a conservative effective length of 0.6s, instead of a theoretical value of 0.5s, where s is the spacing taken as the center-to-center distance between connections. On this basis by substituting $\sigma_e = 1.67f$, $K = 0.6$, $L = 5$ and $r = t/\sqrt{12}$ into the Euler column equation, $\sigma_e = \dfrac{\pi^2 E}{(KL/r)^2}$, the formula in Clause 14.6.5(b) is obtained.

Clause 14.6.5(c) ensures satisfactory close spacing to make a row of connections act as a continuous line of stiffening for the flat sheet under most conditions. A possible exception is a relatively narrow unstiffened flat sheet with w/t up to about 20.

The allowable stresses for unstiffened elements are based on a buckling stress computed from a buckling coefficient of k = 0.5. If an outstanding flange were ideally simply supported (hinged) at the web, it would have a buckling coefficient of 0.425 and would buckle in half-wave equal to its full length. The chosen coefficient of 0.5, therefore, corresponds to a slight rotational restraint of the unstiffened element along its supported edge and to a correspondingly smaller half-wave length. Without detailed investigation the accuracy of which would be somewhat fictitious, this length can be assumed as being not less than 6w. In order for an intermittently connected line to act as one continuous stiffening, at least two connections should be located within one half-wave.

It is this consideration which has led to the provision of Clause 14.6.5.(c) which stipulates that for unstiffened elements connections should be made at distances not exceeding 3w. For large w/t ratios, Clause 14.6.5(b) will automatically take care of this. Hence Clause 14.6.5(c) governs only for relatively narrow unstiffened elements.

The limiting flat width below which failure occurs by yielding and above which it occurs by local buckling is $w = 165t/\sqrt{F_y}$. Corresponding to this condition Clause 14.6.5(c)(i) stipulates a maximum permissible spacing equal to three times this amount, i.e.

$$s = 500t/\sqrt{F_y}$$

but s need not be less than 3w.

14.7 Bracing

14.7.3 Channels and Z-Sections Used as Beams

If channels and Z-sections are used singly as beams they must be braced at intervals to prevent twisting and lateral deflection.

Because sheathing, attached to channels and zees in roofs and walls, provides some restraint against lateral displacement of the connected flange and torsional displacement of the beam, forces are generated in the sheathing from the tendency to lateral and/or torsional displacement. The transfer of these forces into a sufficiently stiff member of the framing may be accomplished by:

1. A sufficiently rigid diaphragm with a mechanism to transfer the forces to the supporting structure;

2. Arranging equally loaded pairs of members to face each other, hence the opposing lateral forces generated by each member are cancelled;

3. Balancing opposing forces in the sheathing materials at a ridge;

4. A system of sag members (rods, angles or channels).

Designers may select other ways to resist these forces and to accomplish the required results.

The fastening system used to attach the sheathing material to the beams must be capable of transferring the lateral forces, and if included in the design, the torsional forces.

14.7.3.2 Bracing When Both Flanges are Braced by Deck or Sheathing Material

When both flanges are connected to deck or sheathing material so as to effectively restrain lateral deflection of each connected flange no other bracing is required.

14.7.3.3 Bracing When One Flange is Braced By Deck or Sheathing Material

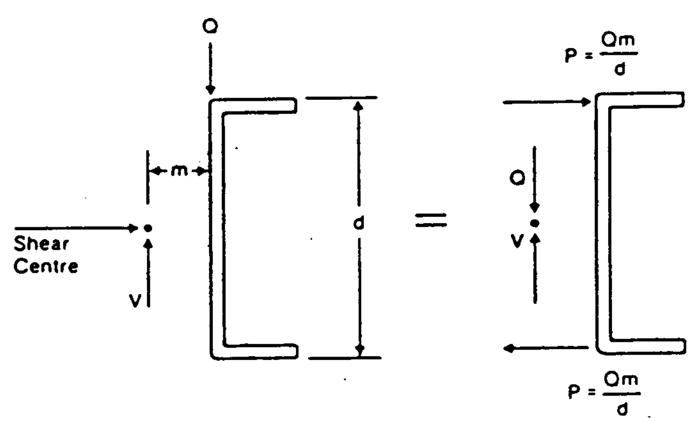
Application of certain loading arrangements, like uplift wind loads and beams continuous over the support, on the unattached flange may induce compression forces. The compression resistance may be increased, over that calculated by assuming the compression flange is a column with pinned ends at points of lateral bracing, by considering the contribution of the web which provides elastic lateral support.[179,180] Research has shown the rotational flexibility of the joint between the beam and the deck or sheathing material to effect the compression resistance.

14.7.3.4 Bracing When Neither Flange is Braced by Deck or Sheathing Material

Special discrete bracing must be provided at specific intervals when neither flange is braced by deck or sheathing material. If all loads and reactions on a beam are transmitted through members framing into the section such that member rotation and lateral displacement are restrained, as frequently occurs in end wall rafter framing of metal buildings, no braces are required.

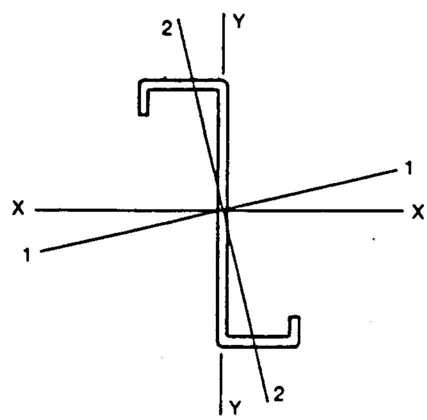
To prevent channel beams from rotating about the shear center special braces must be provided at certain intervals such that additional forces due to the rotation tendency are small enough so the load carrying capacity of the channel is not significantly reduced, and the rotations are kept small, in the order of 1 to 2 degrees. By placing at least three equidistant braces between end supports these requirements are met[181]. However, when a large portion of the total load of the beam is concentrated over a short portion of the span an additional brace must be placed at the load location. The brace must have the strength to prevent the channel from rotating. In Figure 38 the load Q, applied through the plane of the web and not through the shear center, produces a torque. Bracing to resist this torque can be designed for a force $P = Qm/d$. The reactions the brace is required to resist at each flange are given in Clause 14.7.3.4.2.

Figure 38



Since Z-sections are point-symmetrical, the centroid and shear center coincide at the mid-point of the web. A load applied in the plane of the web has no lever arm about the shear center, therefore the same type of rotation that occurs in a channel is not produced. In Z-sections the principal axes are oblique to the web (see Figure 39). A load applied in the plane of the web, resolved in the direction of the two axes, produces both horizontal and vertical deflections. If the horizontal deflection, Δ_x, is permitted to occur, then load P moves sideways and will no longer be in the same plane as the end reactions, thus producing a twisting moment $\Delta_x P$ (see Figure 39). From a series of tests carried out on three different Z-shapes intermittently braced[182] it can be shown that it is only necessary to apply a fictitious horizontal load $P = P(I_{xy}/I_y)$.

Figure 39



14.7.4 Laterally Unbraced Box Beams

Box beams are laterally more stable than single-web sections having the same depth/width ratio. Reference 179 indicates that lateral buckling has no effect on box beams with a steel yield strength of 227 MPa, for a ratio of laterally unsupported length to the distance between webs of the section (L/b) as high as 100. Based on this, the laterally unsupported length shall not exceed 75b for a steel with F_y = 227 MPa. Therefore, in Clause 14.7.3 the box section can be used without taking into account lateral buckling as long as the length does not exceed 17 000F_y.[183]

When the closed side of a hat section is in compression it is more stable against lateral buckling. Although no specific provisions for the design of unbraced hat sections are given the following have been used in design.[179]

Taking the y-axis as the axis of symmetry and applying the bending moment about the x-axis of the hat section then:

(a) When $I_y > I_x$, no reduction in resistance (LSD) or allowable stress (WSD) is required for lateral buckling.

(b) When $I_y < I_x$, the bending resistance (LSD) or allowable bending stress (WSD) can be conservatively determined using the equation for slender columns.

In the above L is the unsupported length and r_y is the radius of gyration of the compression portion of the hat section about the y-axis.

14.8 - Testing

This Clause outlines the tests and procedures for establishing the mechanical properties, which are required by the designer using Clause 14. These properties must be established by test and certified by a professional engineer.

Clause 14.8.2.2 makes provision for proof of structural adequacy by load testing. It must be emphasized that this clause is only intended to apply for members and/or assemblies which, because of shape or type, load capacity and deflection, cannot be calculated within the provisions of Clause 14.

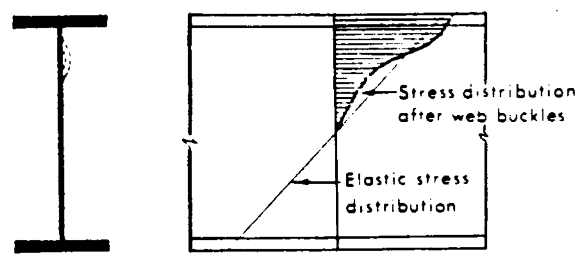
15. FATIGUE

For the few cases where members and connections of building structures are subject to the types of repeated loading which gives rise to fatigue conditions, Clause 15 provides the requirements to design these members and connections for the appropriate fatigue life. The Standards define fatigue as an ultimate limit state but one for which the requirements are checked at specified load levels. However, Clause 15.1.1 requires that factored resistances be sufficient for the factored static loads in addition to the fatigue requirements of Clause 15.

The "stress range" concept reflects the results of a comprehensive research project, [67,68,69] which states that the difference between the maximum applied stress and the minimum applied stress (the stress range) accounts for nearly all the variation in fatigue life for a given steel member or detail.

Figure 40

APPROXIMATE STRESS DISTRIBUTION IN GIRDERS
WITH BUCKLED WEB



Clause 16.3 accommodates the extra flange load by reducing the moment resistance of the girder, in LSD or the allowable bending stress in WSD. In both cases, an elastic linear stress distribution is assumed. In most cases the applicable reduction is small. Figure 41 (LSD) and Figure 42 (WSD) illustrate the amount of reduction according to the formula given in the clause.

The limit of $1810/\sqrt{F_y}$ on the slenderness of a Class 3 web is replaced in this clause by $1810/\sqrt{M_r/(\phi S)}$ in LSD and $1810/\sqrt{F_{bc}}$ in WSD to account for the possibility that M_r in LSD or F_{bc} in WSD may already have been reduced in Clause 13.5 or 13.6 by consideration of flange instability (lateral flange buckling). The effect of the substitution is to increase the slenderness at which the reduction will be imposed; this is justified by the lower compressive bending stresses present in

Figure 41

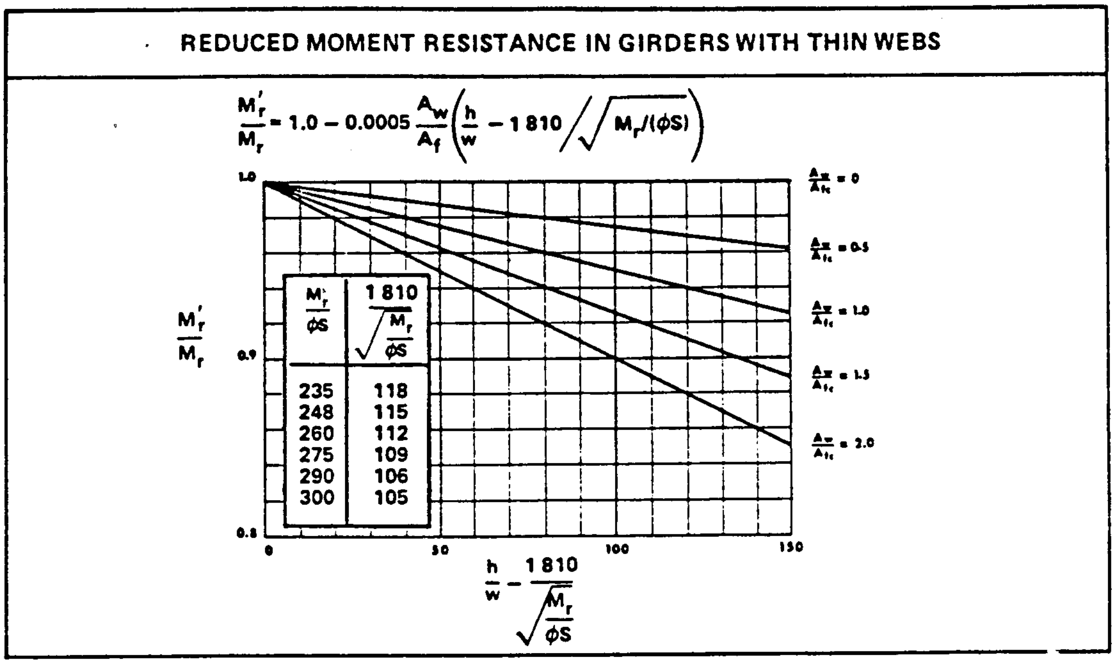
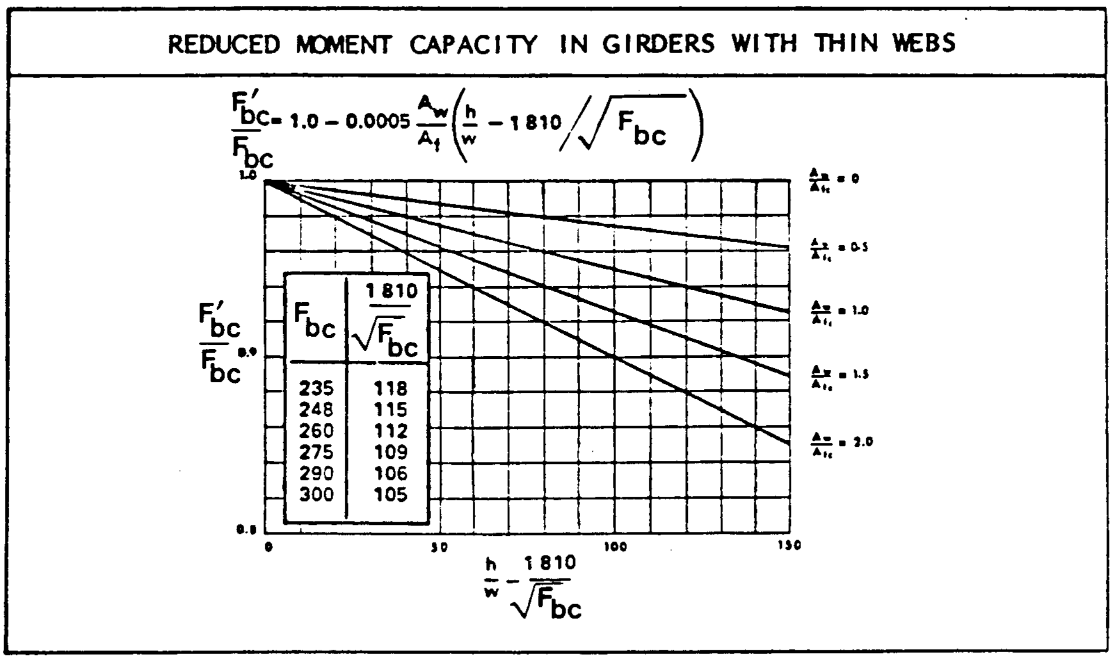


Figure 42



the web and the consequently decreased possibility of web buckling sideways.

No data are available to indicate that a partial plastic hinge can develop in a girder with a Class 4 web and therefore in LSD the moment is limited to ϕM_y and in WSD the allowable stress is limited to 0.60 F_y.

16.4 Flanges

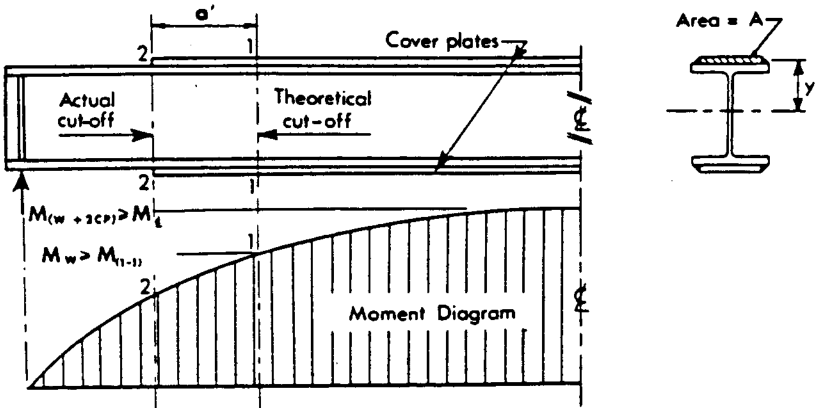
Clause 16.4.4 requires partial length cover plates to extend sufficiently beyond the theoretical cut-off point such that the horizontal shear force at the theoretical cut-off point can be developed by the fastener or welds and transmitted to the cover plate. In the case of welded cover plates, theoretical and experimental studies[58] have shown that the horizontal shear force ($\frac{AM_f y}{I_g}$ in LSD or $\frac{AM_c y}{I_g}$ in WSD) is developed within a specific length (a') from the actual end of the cover plate. Because the maximum weld size is limited by the cover plate thickness, it may not be possible to develop the shear force, P, at the theoretical cut-off point. Then the cover plate length must be increased such that the smaller shear force a length a' away from the end of the cover plate can be developed by the terminal welds. Figure 43 illustrates this requirement.

16.6 Intermediate Transverse Stiffeners

Spacing the intermediate transverse stiffeners to suit the shear resistance of Clause 13.4 permits the proper tension fields to occur in the web panels under load. However, since a panel

Figure 43

COVER PLATE DEVELOPMENT



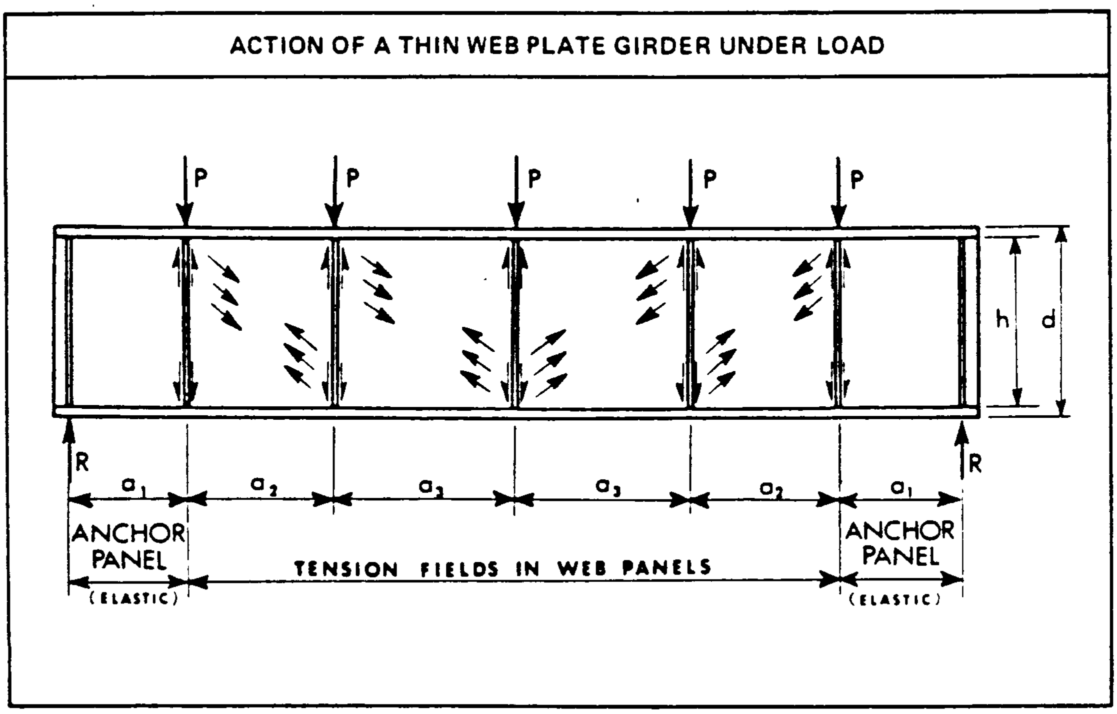
Force to be developed in cover plate within distance a' is $\frac{AM_{(1-1)}y}{I_{(w+2CP)}}$

For welded cover plates a' is not to exceed lengths specified in Clause 16.4.4. In certain cases this will necessitate an increase in weld size or an extension of cover plate so that force at distance a' from its end equals that which terminal welds will support.

with a large opening cannot develop a tension field, it, as well as the end panels which serve to anchor the truss-like action, must remain elastic. Thus, the smaller panel dimension, a or h, is limited to 1150 $w/\sqrt{V_r/\phi A_w}$ in LSD or 890 $w/\sqrt{f_v}$ in WSD. This expression is derived directly from Clause 13.4.1(d) for $\tau = 1.0$, $\eta = 0$ and $k_v = 7.34$[42]. Figure 44 illustrates the action of a thin web girder under load.

When intermediate transverse stiffeners are required, Clause 16.6.2 puts limits on the maximum spacing. Thus, the maximum spacing may be limited by this clause or by shear resistance of the web. Insofar as Clause 16.6.2 is concerned, when $h/w < 150$ the ratio a/h cannot exceed 3. When $h/w > 150$, the

Figure 44



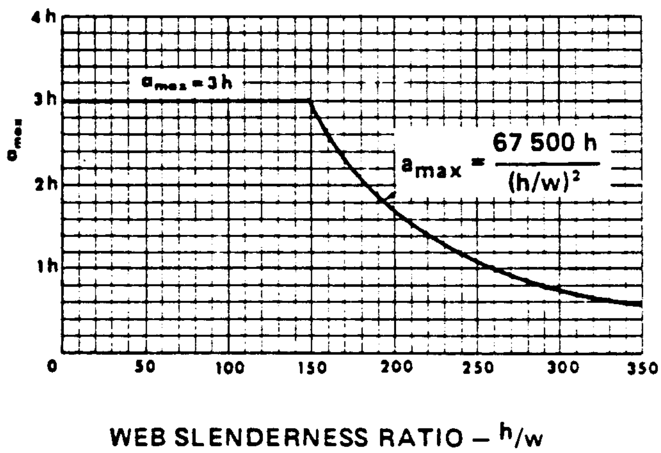
ratio a/h cannot exceed $\frac{67,500}{(h/w)^2}$. Figure 45 shows the relationship graphically. When a/h exceeds 3 the value of the stiffener is very minor, whereas when h/w > 150 the maximum stiffener spacing is reduced for ease in fabrication and handling.

Clause 16.6.3 requires that intermediate transverse stiffeners have both a minimum moment of inertia and a minimum area. The former provides the required stiffness when web panels are behaving in an elastic manner; the latter ensures that the stiffener can sustain the compression to which it is subjected when the web panel develops a tension field. Since stiffeners

Figure 45

MAXIMUM STIFFENER SPACING

MAX. INTERMEDIATE TRANSVERSE STIFFENER SPACING-'a' IN TERMS OF WEB DEPTH-'h'



Note: Stiffener spacing 'a' may need to be less than the max. permissible value if ultimate web shear stress is high, also at end panels and panels containing large web openings.

subject to compression act as columns, stiffeners placed only on one side of the web are loaded eccentrically and are less efficient. The stiffener factor (D) is included in the formula for stiffener area to account for the lowered efficiency of stiffeners furnished singly rather than in pairs.

Clause 16.6.4 requires that a minimum shear be transferred between the stiffener and the web.[42] Intermediate stiffeners furnished singly (rather than in pairs) must be attached to the compression flange so as to assist in preventing the flange from tipping under loads.

16.8 Web Crippling

The equations given in this LSD clause provide a safe estimate of the factored compressive resistance of webs of rolled beams and welded plate girders. They are derived from the equations given in the WSD Standard by multiplying those equations by the nominal factor of safety of that Standard, 5/3. Experience has shown that the working stress equation has provided a satisfactory margin of safety[59]. In fact Reference 60 recommends that the compressive resistance be computed as the yield stress acting on the web over a length equal to the length of bearing plus 5k. For long bearing lengths (15k) the difference in compressive resistance for interior loads computed by Reference 60 and that by Clause 16.8 is negligible.

16.9 Stability of Thin Webs

In Reference 61 it is suggested that the critical buckling loads for webs, based on elastic analysis of the buckling of plates, can be computed by

$$B_{cr} = \frac{\pi^2 E}{12(1-\nu^2)(h/w)^2} \left(5.5 + \frac{4}{(a/h)^2} \right) A$$

or

$$B_{cr} = \frac{\pi^2 E}{12(1-\nu^2)(h/w)^2} \left(2 + \frac{4}{(a/h)^2} \right) A$$

for the two cases when the flange is or is not restrained against rotation, respectively. Flange restraint would be provided, for example, when a concrete slab is cast on top of the beam.

For steel, the quantity $\pi^2E/12(1-\nu^2)$ is equal to 180 760 and the value of 115 000 in equations of LSD Clause 16.9 provides an additional margin of about 1.6 to take into account the vulnerability of light webs to instability. This also provides for the possibility of overloading of a small floor area which, over the whole area, is loaded only to the specified value. Figure 46 illustrates the application of these LSD formulas.

The WSD Clause 16.9 for critical buckling stress has a coefficient of 69 000 which reflects the application of the nominal factor of safety of 5/3 used in this WSD Standard. Figure 47 illustrates the application of the WSD formulas.

16.10 Openings

In general, if the factored moments or shears of LSD or the computed bending or shear stresses of WSD at the net section exceed the capacity of the member at that point, reinforcement is required. However, research[62] has indicated that unreinforced circular openings may be used in prismatic compact beams or girders without a reduction in their resistance under the conditions listed in Clause 16.10.2 Figure 48 illustrates the geometrical conditions of Clause 16.10.2.

The analysis to account for the effect of the opening on a member may be either elastic[63,64] or plastic[63,65,66]. In the case of plastic analysis the flanges may meet Class 2 requirements but the web must meet the requirements of Class 1 sections.

Figure 46

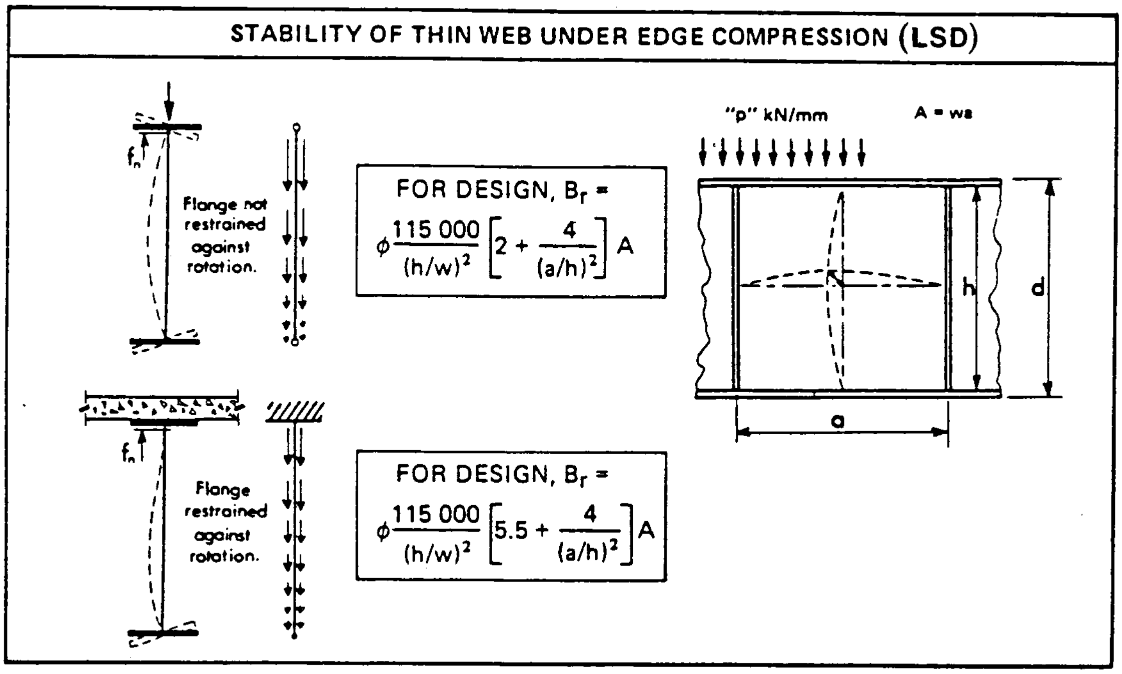


Figure 47

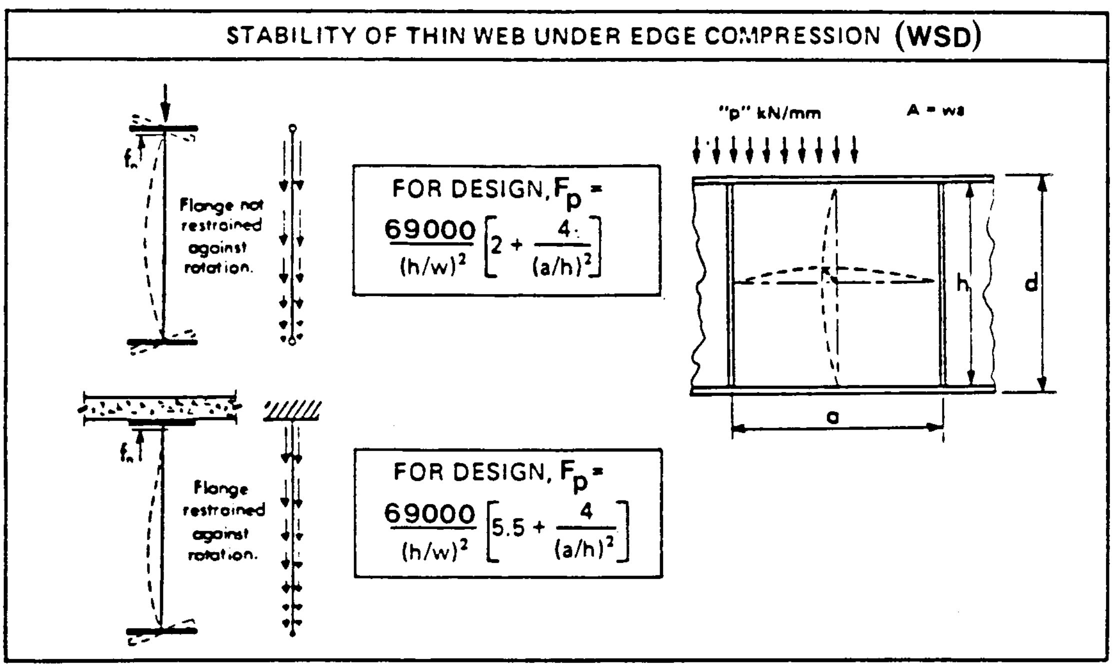
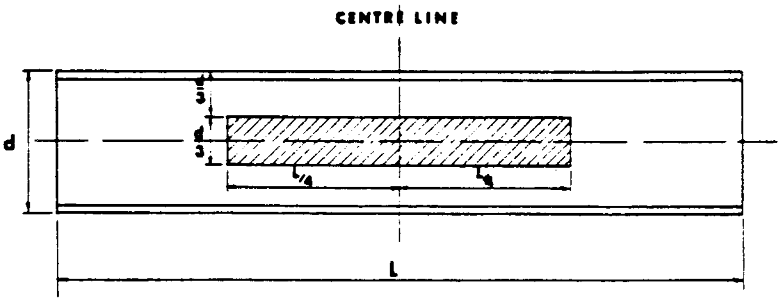


Figure 48

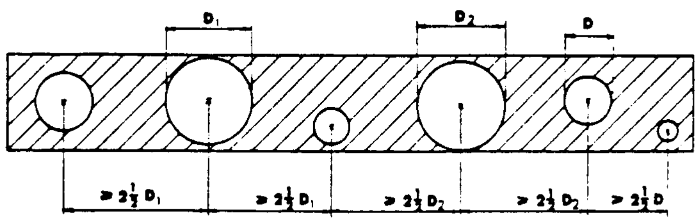
UNREINFORCED CIRCULAR WEB OPENINGS IN BEAMS



Unreinforced circular holes may be placed anywhere within the hatched zone without affecting the strength of the beam for design purposes, provided:

1. Beam supports uniformly distributed load only.
2. Beam section has an axis of symmetry in plane of bending.
3. Spacing of holes meets the requirements shown below.

SPACING OF HOLES



Spacing must be > 2½ times diameter of the larger opening of any two adjacent openings.

16.11 Torsion

In many cases beams are not subject to torsion because of the rėstraint provided by slabs, bracing or other framing members. However, when the forces and moments due to torsion must be resisted by the members, it is in addition to any other moments and forces to which the member is subjected. For guidance, References 70 and 71 should be consulted.

17. OPEN-WEB STEEL JOISTS

17.1. Open-web steel joists (OWSJ) are generally proprietary products whose design, manufacture, transport and erection are covered by the requirements of Clause 17. The Standard clarifies the information to be provided by the building designer (user-purchaser) and the joist manufacturer (joist designer-fabricator).

17.2 Definitions

There are many variations of the simply supported joist which has given rise to the definition for **special open-web joists or special joists.** These joists are those subjected to **specific loading conditionsl,** cantilever joists, continuous joists and joists having special support conditions. Span is the distance centre-to-centre of joist bearings and may be any length.

17.3 Materials

The use of yield strength levels reported on mill test certificates for the purposes of design is prohibited as this practice may significantly lower the margin of safety by not properly accounting for the statistical distribution of yield levels. Historically all design rules have been, and still are, based on the use of the **specified** minimum yield point or yield strength.

17.4.1 Building Design Drawings

The Standard recognizes that the building designer may not be the joist designer; therefore, building design drawings are required to provide specific information for the design of the joists. Loads such as unbalanced, non-uniform, concentrated, and net uplift, are to be shown by the building designer. Options, such as attachments for deck when used as a diaphragm, special camber and any other special requirements should also be provided. Although steel joist manufacturers may indicate the maximum clear openings for ducts, etc. which can be accommodated through the webs of each depth of their OWSJ, building designers should, in general, show on the building design drawings the size, location and elevation of openings required through OWSJ. Large ducts may be accommodated by special design. Ducts which require open panels and corresponding reinforcement of the joist, should where possible, be located within the middle half of the joist. This information is required prior to the time of tendering to permit proper costing.

17.4.2 Joist Design Drawings

A joist manufacturer's design information may come in varying forms including: design sheets, computer printout, tables, etc. Not all joist manufacturers make "traditional" detail drawings.

17.5 Design

17.5.1 and 17.5.2 Loading

Both OWSJ's and special OWSJ's are to withstand three separate loading conditions:

(1) uniformly distributed load;

(2) the unbalanced load which would produce the most critical effect on any component; and

(3) an appropriate concentrated load.

For special OWSJ's, Table 1 gives concentrated loads that may be used.

To accommodate load tables in OWSJ catalogues for such purposes as estimating, the Standard provides for specific unbalanced and concentrated loading conditions.

Since the building designer specifies the loading conditions, the design loading for joists should be shown clearly on the drawings. A joist schedule (Figure 49) can be used to record all loads on joists.

On a joist schedule specific joist designations from a manufacturer's catalogue or from the AISC and Steel Joist Institute in the U.S.A. are not appropriate and should not be

Table 1

Loading Requirements for Special OWSJs	
Area of Floor or Roof	**Minimum Specified Concentrated Load, kN**
Roof surfaces	1.3
Floors of classrooms	4.5
Floors of offices, manufacturing buildings, hospital wards and stages	9.0
Floors and areas used by passenger cars	11
Floors and areas used by vehicles not exceeding 3,600 kg gross mass	18
Floors and areas used by vehicles exceeding 3,600 kg but not exceeding 9,000 kg gross mass	36
Floors and areas used by vehicles exceeding 9,000 kg gross mass*	54
Driveways and sidewalks over areaways and basements*	54

* Special study is required to determine concentrated loads for the design of floors and areas used by vehicles exceeding 9,000 kg gross mass and driveways and sidewalks over areaways and basements.

specified. All heavy concentrated loads such as those resulting from partitions, large pipes, mechanical and other equipment to be supported by OWSJ, should be shown on the structural drawings. Small concentrated loads may be allowed for in the uniform dead load.

Because the depth of the joist supplied by different joist manufacturers, may vary slightly indicate in the remarks column when specified depth is critical. When the importance factor, γ, is not equal to 1.0 it should be specified by the building designer. It is recommended that the building designer insert a suggested moment of inertia, I_x, in the remarks column where vibration of a floor system is a consideration.

the deck tends to effect "panel point" loading of the joist, thus, offsetting the theoretical reduction in chord capacity. When the panel length exceeds 610 mm, a simplified form of the beam-column interaction[73] formula is used. When calculating bending moments in the end panel, it is customary to assume the end of the chord to be pinned even though the joist bearing is welded to its support. Also, when determining the appropriate width-thickness ratio of the compression top chord, the stiffening effect of supported deck or the web is to be neglected (Clause 17.5.6.1).

17.5.9 Webs

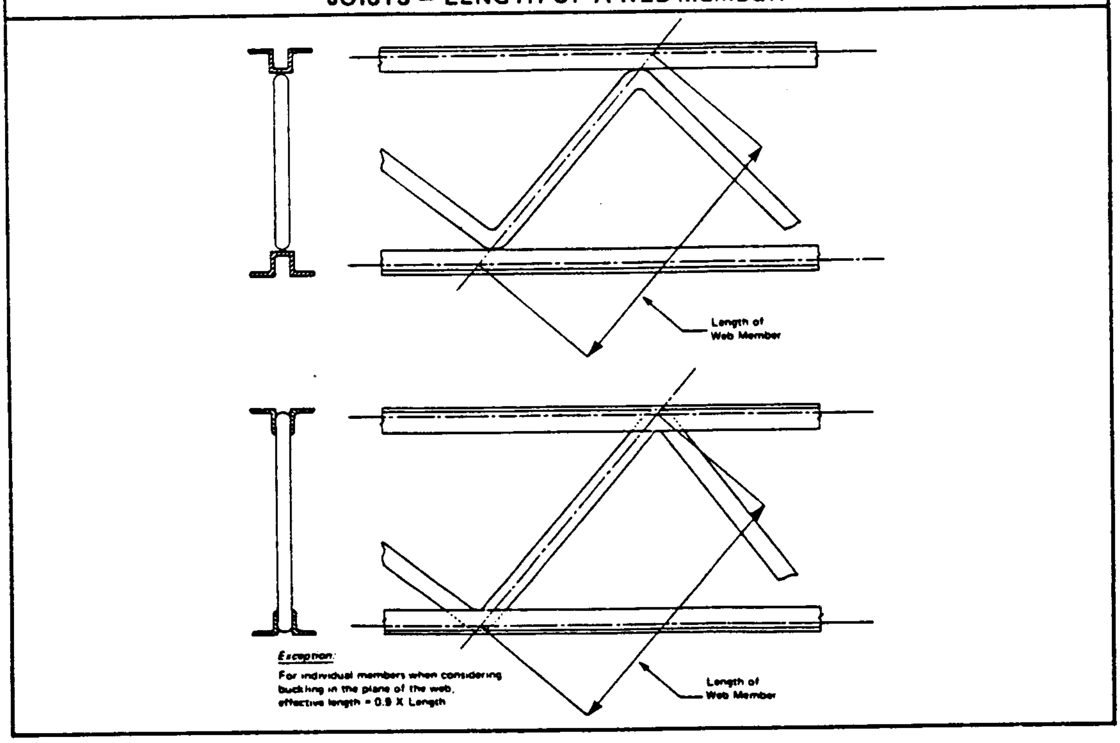
The length of web members for design purposes are shown in Figure 50. With the exception of web members made of individual members, the effective length factor is always taken as 1.0. For individual members this factor is 0.9 for buckling in the plane of the web, but is 1.0 for buckling perpendicular to the plane of the web.

Web members in tension are not required to meet a limiting slenderness ratio, which is significant when flats are used as tension members. However, attention should be paid to those loading cases where the possibility of shear reversal must be considered. It is likely that tension diagonals (except for end diagonals) may have to resist compression forces due to requirements for concentrated loads.

The Standard requires the joist to be checked for a critical unbalanced loading condition.

Figure 50

JOISTS – LENGTH OF A WEB MEMBER



17.5.10 Spacers and Battens

Spacers and battens can only be omitted when the least radius of gyration of each section is equal to or more than the least radius of gyration of the built-up member. "Integral part" means that an additional element attached to the joist during construction, such as the steel deck, may not serve as a spacer or batten. [Clause 17.5.8.(c)]

17.5.11 Connections and Splices

Splices are permitted at any point in chord or web members. In LSD the splices must have a factored resistance greater than the factored load effect and in WDS the stresses due to design loads must be less than the proper allowable stresses. As a general rule, it is preferable to have the gravity axes of members meet at a common point within a joint. However, when this is not practical, eccentricities may be neglected if they do not exceed those described in Clause 17.5.11.4 (Figure 51). A research project at the University of Alberta has shown that the effect of small eccentricities which are present in joists to be of a minor consequence except for eccentricities, at the end bearing and the intersection of the end diagonal and bottom chord[119]. (See also Clause 17.5.12.4.)

17.5.12 Bearings

Special attention is drawn to the possibility of eccentricity between the centre of bearing and the intersection of the axes of the chord and the end diagonal. Since the location of the centre of bearing is dependent on the field support conditions, and construction tolerances, it may be wise to assume a maximum probable distance centre-to-centre of bearing when designing the bearing detail. Lacking specific information a reasonable assumption might be the overall length of the joist less 65 mm (since the minimum bearing on a steel support is 65 mm). Therefore, the location of the centre of bearing would be 32 mm from the end of the joist (Figure 52). When detailing the joists, care must be taken to provide sufficient clearance between the end diagonal and the supporting member or wall

Figure 51

JOISTS – ECCENTRICITY LIMITS

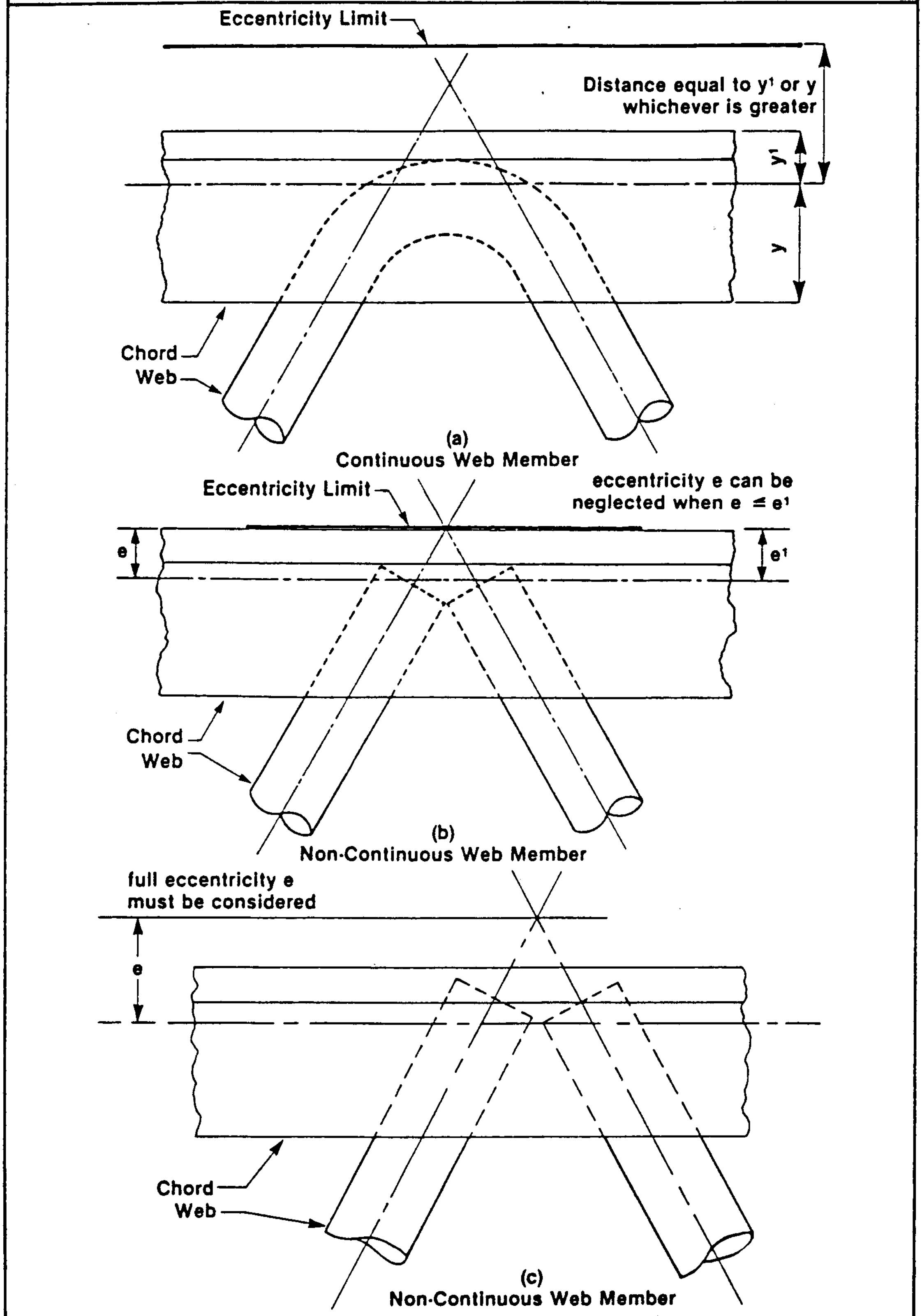


Figure 52

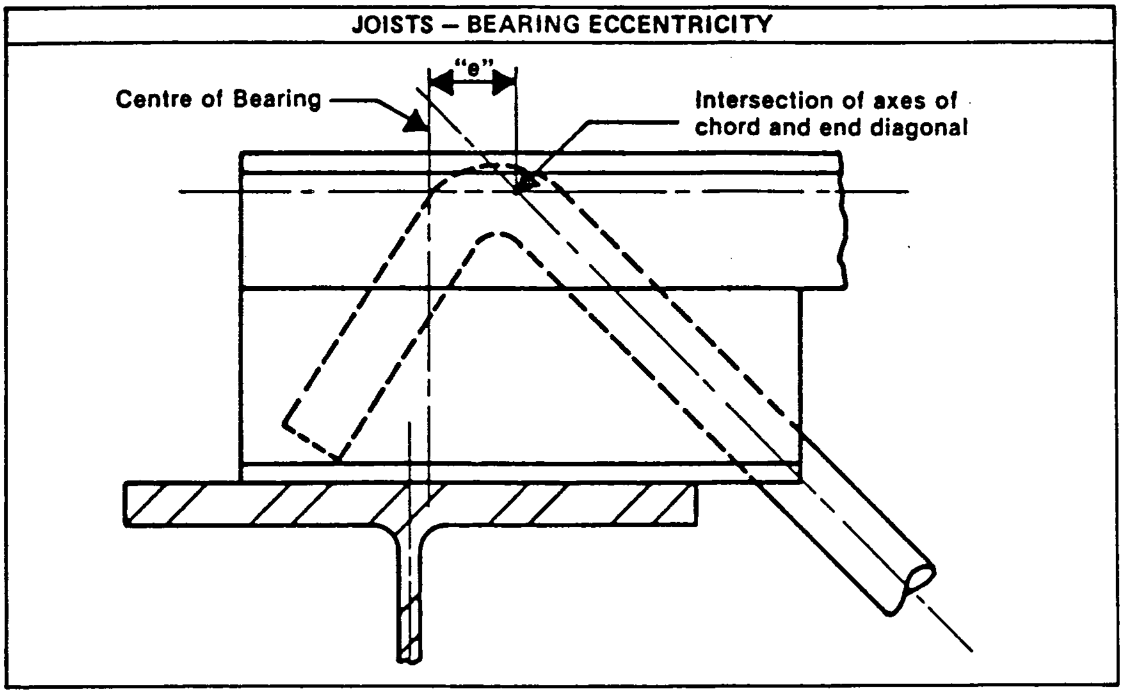


Figure 53

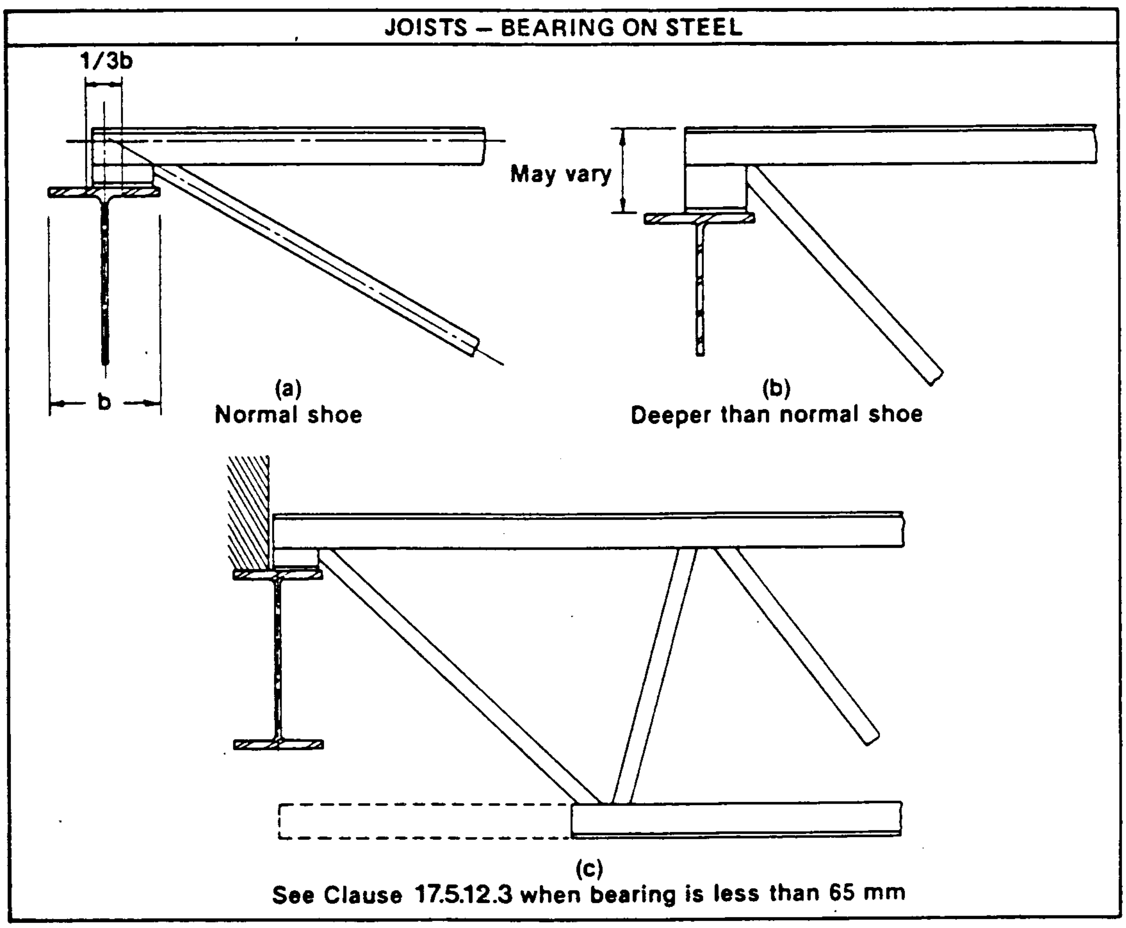
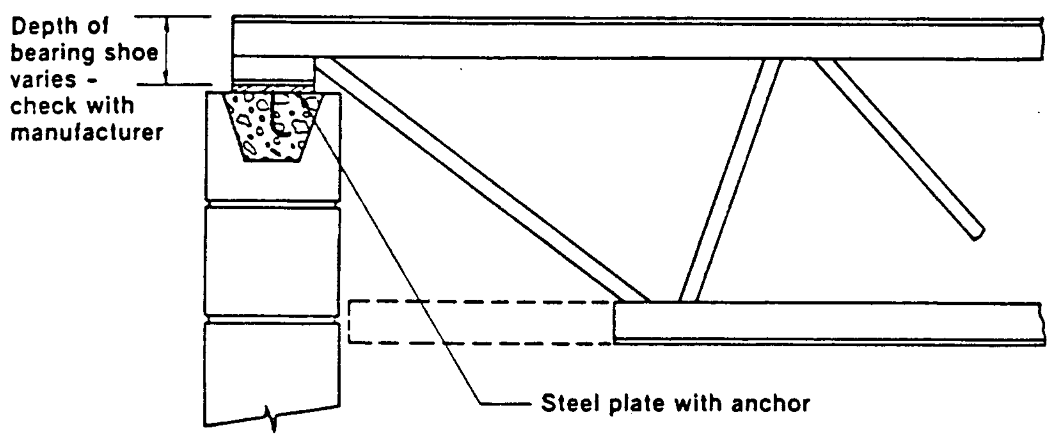


Figure 54

JOISTS – BEARING ON STEEL PLATES ANCHORED TO CONCRETE AND MASONRY



(maximum 25 mm). One solution, to obtaining proper bearing, is to increase the depth of the bearing shoe.

If the support is found to be improperly located, such that the span of the joist is increased, the resulting eccentricity may be greater than that assumed. Increasing the length of the bearing shoe to obtain proper bearing may create the more serious problem of increasing the amount of eccentricity. The Standard requires that the bearing detail and the end of the joist be proportioned to include the effect of such eccentricity. Minimum bearing area on concrete and masonry should be based on the bearing resistance or capacity of those materials. The minimum length of bearing on concrete or masonry is 80 mm to avoid interference between the ends of the joists and the outside course of brick or a 200 mm wall. On steel 65 mm is desirable but not mandatory; however, there has to be enough bearing to get minimum anchorage.

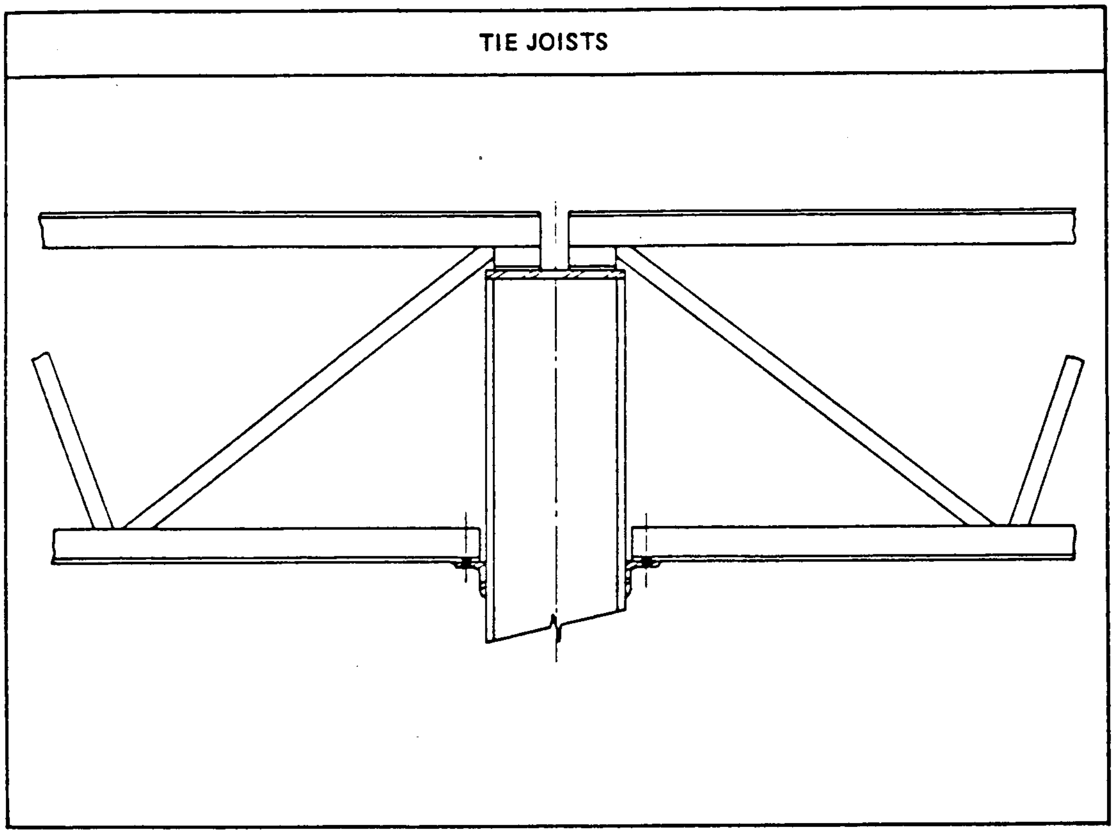
17.5.13.2 Tie Joists

The function of tie joists is to assist in the erection and plumbing of the steel frame. Plumbing cables should be so installed that excessive compressive forces are not introduced into the joists. Top and bottom chords are both attached at the column line. This may be done by bolting either the top or bottom chord; after plumbing the columns, the other chord is then usually welded (Figure 55).

Since the joists, by definition, are only to aid in the erection, spacing and plumbing of columns, the ends of at least one chord of these joists must be detailed for connection by mechanical fasteners. Either the top or the bottom chord may be used for this purpose. In most buildings, tie joists remain as installed with both top and bottom chords connected; however, current practices vary with, in some cases, the bottom chord connections to the columns being made with slotted holes. A research project at McGill University studied the behaviour of tie joist connections and concluded that tie connection may be insufficient to carry lateral loads[120].

For spandrel beams and other beams on which joists frame from one side only, the centre of bearing shoe should be located

Figure 55



within the middle third of the flange of the supporting beam. By using a deeper than normal shoe, interference between the support and the end diagonal may be avoided.

17.5.13.3 Frame Action

Where frame action involving joists is desired, the appropriate moments and forces are to be shown on the building design drawings to enable the joist and the joist-to-column connections to be designed by the joist manufacturer.

If joists are to be used to brace columns or to resist lateral forces on the finished building, the appropriate axial forces, moments and shears should be noted on the structural drawings to facilitate the proper design of these special joists.

17.5.4 Deflections

Deflections are to be computed under specified load levels. The method of computing deflections given in Clause 17.5.14.2 has been verified by tests[73].

17.5.15 Camber

Nominal camber can be in accordance with the values tabulated in Table 2 according to the expression in Clause 17 5.15 rounded to the nearest millimetre, or as specified by the building designer. Specific manufacturing tolerances (both plus and minus) are covered in Clause 17.10.9 and are applicable to either nominal or specified camber. For joists spanning 17,000 mm or greater, the maximum difference in camber has been set at 20 mm to limit the difference in camber between two adjacent joists.

17.5.16 Vibration

Appendix F is a Guide for Floor Vibrations and contains recommendations for floors using steel joists. By increasing the floor thickness (mass), damping will be increased more efficiently than by increasing the moment of inertia (I_x) of the joists.

Table 2

CAMBER (mm)			
Span	**Nominal Camber**	**Minimum Camber**	**Maximum Camber**
Up to 6 000		3	10
7 000	3	3	11
8 000	4	3	12
9 000	6	3	14
10 000	7	3	16
11 000	8	3	17
12 000	10	3	19
13 000	12	3	21
14 000	14	4	23
15 000	16	6	26
16 000	18	8	28

17.5.17.3 Fabricator and Erector Qualification

A fabricator or erector certified in Division 3 may meet the requirement of this Standard by having the work done under the supervision of a fabricator certified in Division 1 or 2.1.

17.5.17 Welding

Flux and slag are removed from all welds to assist in the inspection of the welds as well as to increase the life of the protective coatings applied to the joists.

17.6 Stability During Construction

A distinction is made between bridging, put in to meet the slenderness ratio requirements for top and bottom chords, and the temporary support required by Clause 17.6 to hold joists against movement during construction. Permanent bridging, of course, can be used for both purposes.

17.7 Bridging

Figures 56 to 58 provide illustrations of bridging and details of bridging connections.

Figure 56

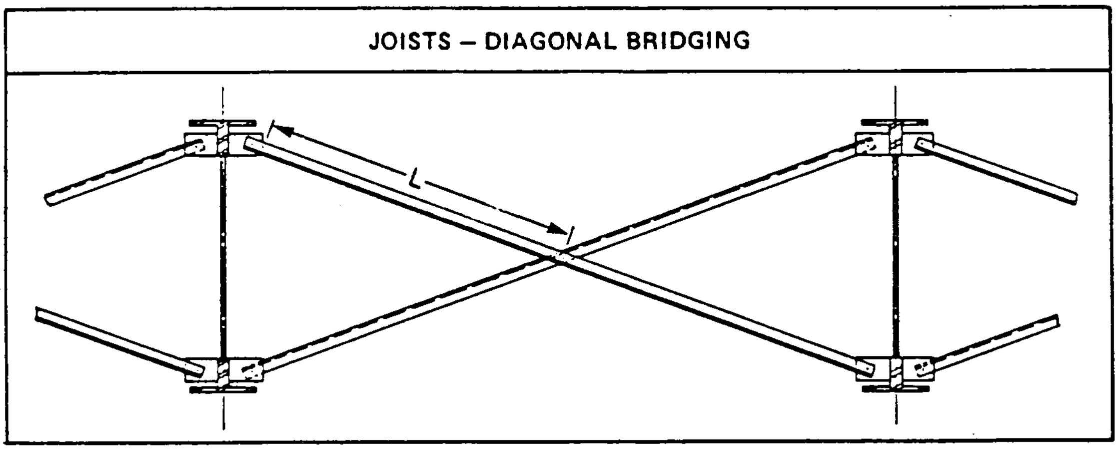
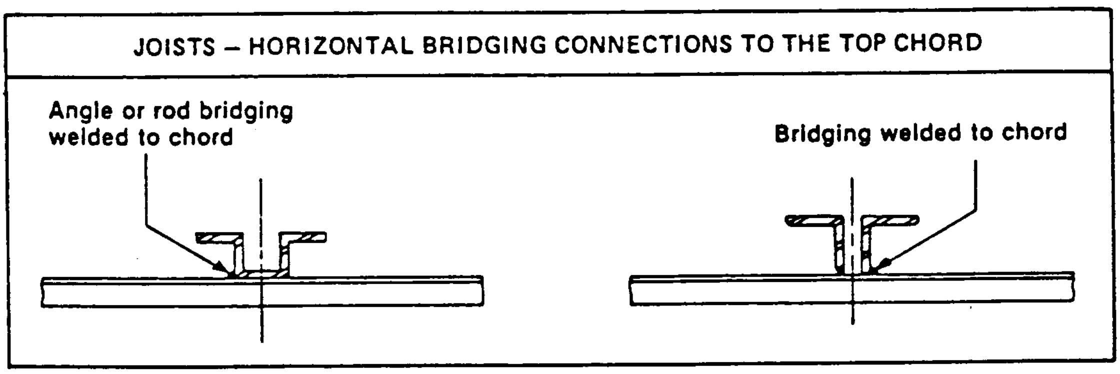


Figure 57



17.7.7 Anchorage of Bridging

Ends of bridging lines can be anchored to the adjacent steel frame or adjacent concrete or masonry walls as shown in Figure 59.

Figure 58

JOISTS – HORIZONTAL BRIDGING CONNECTIONS TO THE BOTTOM CHORD

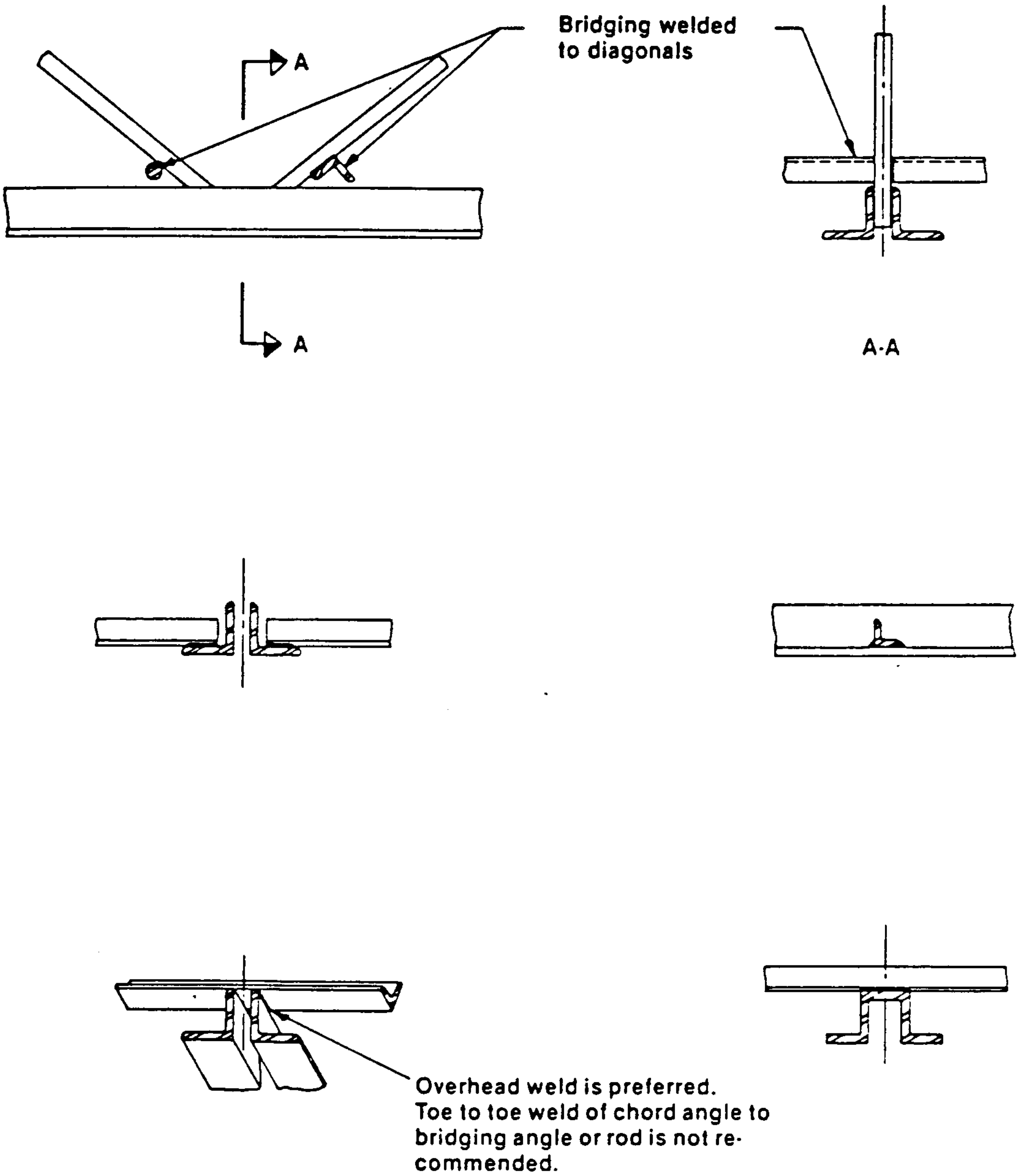
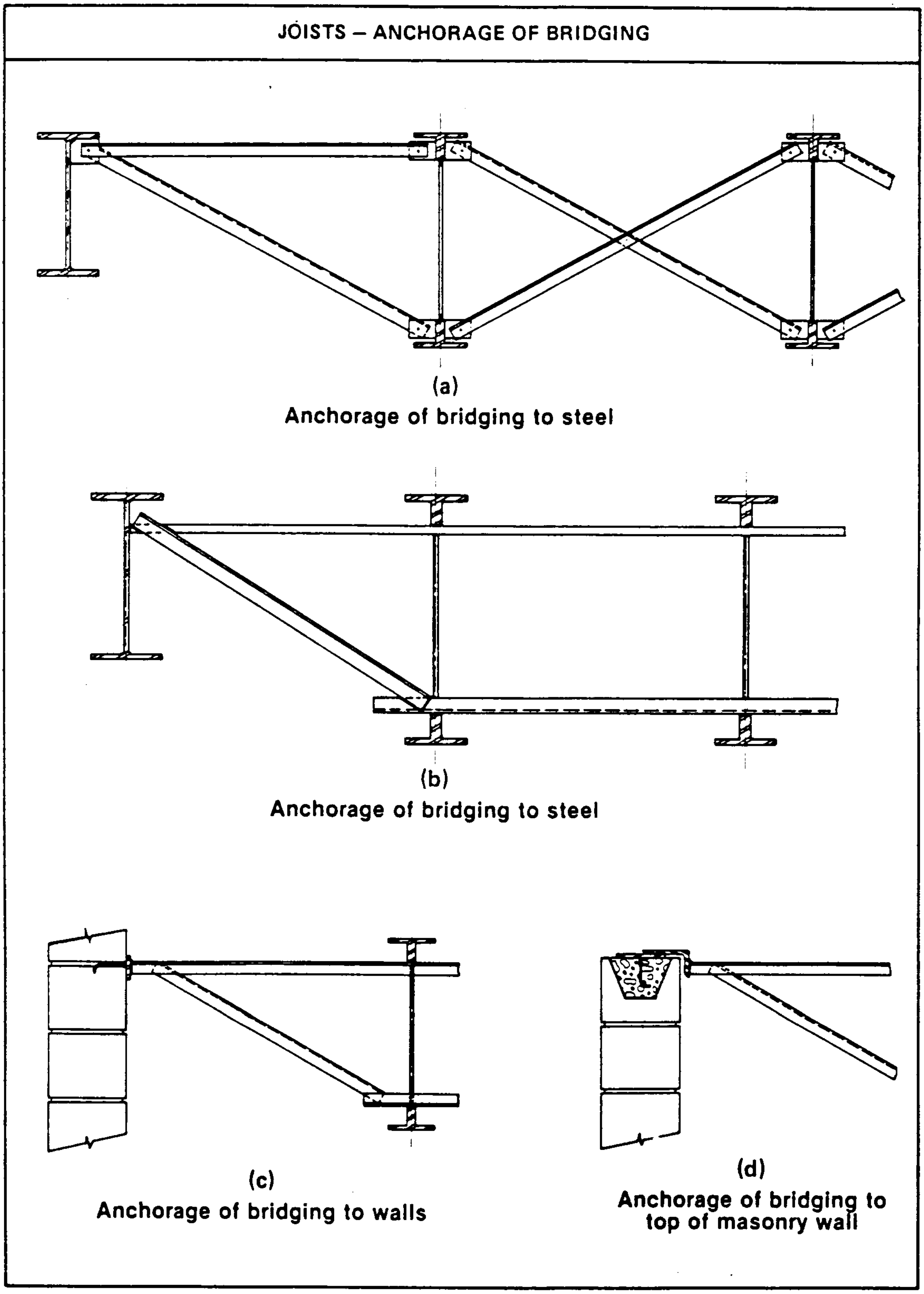


Figure 59


JOISTS – ANCHORAGE OF BRIDGING
(a)
Anchorage of bridging to steel
(b)
Anchorage of bridging to steel
(c)
Anchorage of bridging to walls
(d)
Anchorage of bridging to top of masonry wall

Where attachment to the adjacent steel frame or walls is not practicable, diagonal and horizontal bridging shall be provided in combination between adjacent joists near the ends of bridging lines as shown in Figure 60.

Extra lines of bottom chord bridging may be required, and joists should be checked for stress reversals resulting from the cantilevered end.

It is generally considered good practice to install a line of bridging at the first bottom chord panel point as shown in Figure 61.

17.7.9 Spacing of Bridging

Either horizontal or diagonal bridging is acceptable, although, horizontal bridging is generally recommended for shorter spans up to about 15 000 mm and is usually attached by welding. Diagonal bridging is recommended for longer spans and is usually attached by bolting. Bridging need not be attached at panel points and may be fastened at any point along the length of the joists. When horizontal bridging is used, bridging lines will not necessarily appear in pairs as the requirements for support of tension chords are not the same as those for compression chords. Since the ends of joists are anchored, the supports may be assumed to be equivalent to bridging lines. Joists bearing on the bottom chord will require bridging at the ends of the top chord.

Figure 60

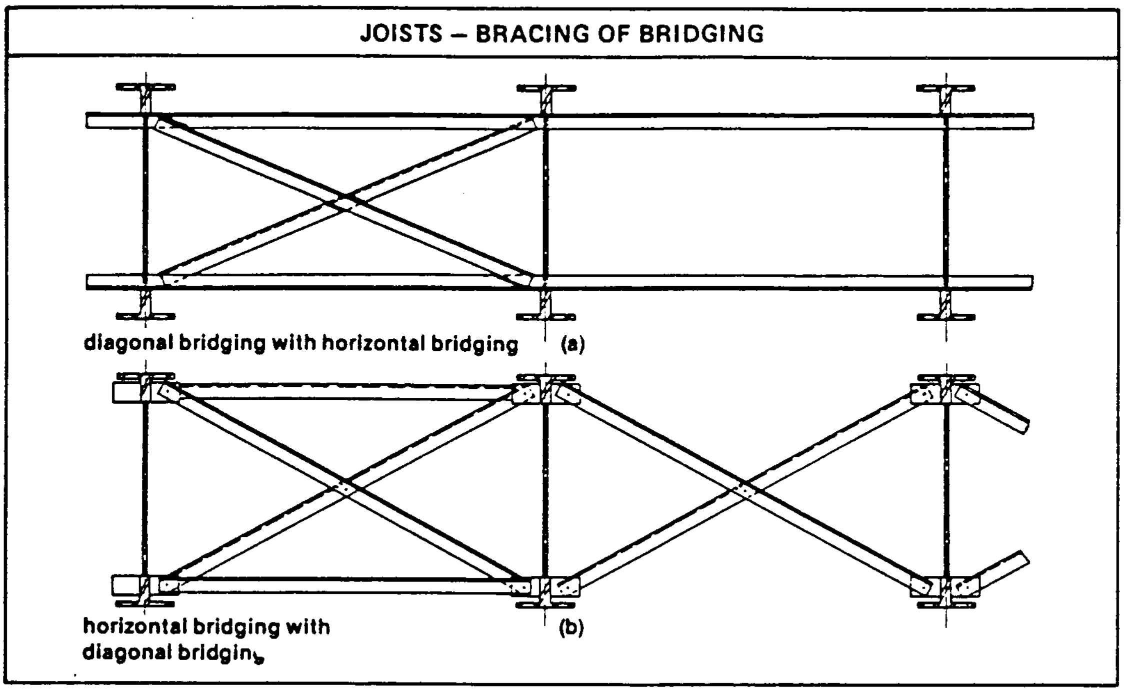
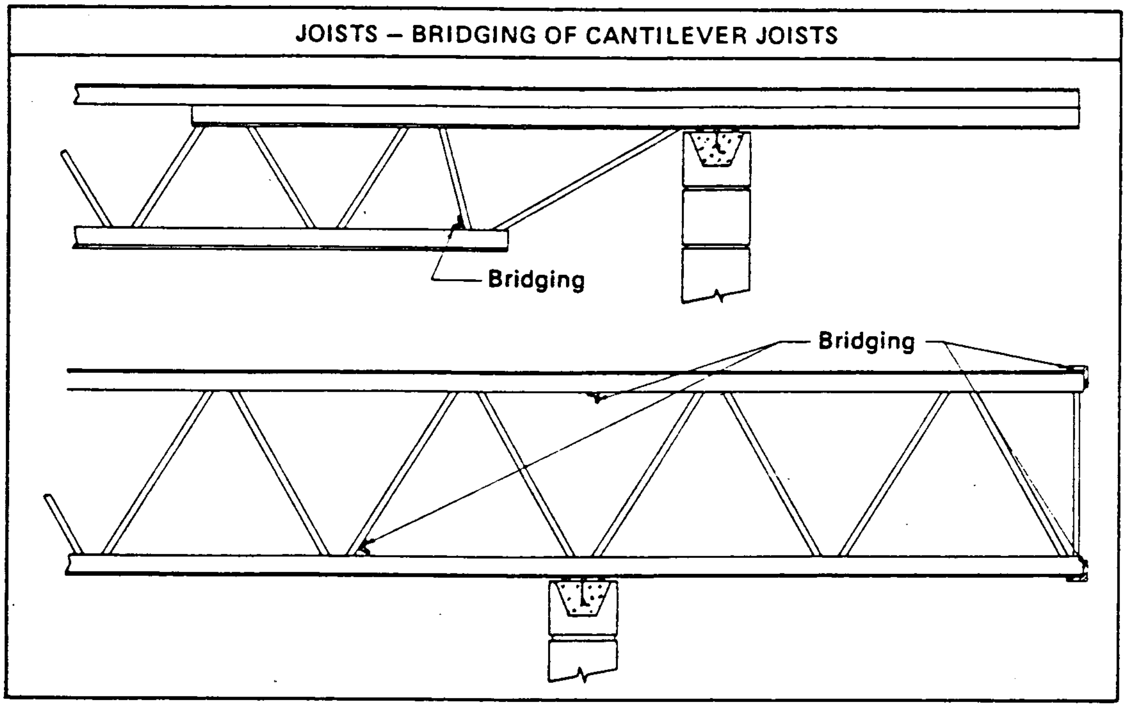


Figure 61



17.8 Decking

Decking is assumed to be sufficiently rigid to provide lateral support to the compression chord of the joist. In these special cases where decking is not capable of furnishing the required lateral support, the compression chord is to be braced laterally in accordance with the rules for providing stability to beams, girders and trusses; that is, be proportioned to resist a force equal to 1 percent of the force in the compression chord at the point of support or a total uniformly distributed load equal to 5 percent of the force in the compression chord.

Where the decking complies with Clause 17.8 and is sufficiently rigid to provide lateral support to the top (compression) chord, the top chord bridging is no longer required. Bottom (tension) chord bridging is permanently required such that the unsupported length of the chord does not exceed 240 r as defined in Clause 17.7.9.

Clause 17.8.3, requires the building designer to show the special attachment requirements on the building design drawings when decking is used in combination with joists to form a diaphragm for the purpose of transferring lateral applied loads to vertical bracing systems.

17.9 Shop Painting

Interiors of buildings conditioned for human comfort are generally assumed to be of a non-corrosive environment and therefore do not require corrosion protection.

OWSJ's normally receive one coat of paint suitable for a production line application. This paint is adequate for three months of exposure, which should be ample time to enclose the OWSJ's in the finished structure and/or finish painting the joists.

Special coatings and paints with special preparations are expensive because they have to be applied individually to each joist by spraying or other means.

17.10 Manufacturing Tolerances

Figure 62 illustrates many of the manufacturing tolerances.

17.11.3 Quality Control

When testing forms part of the manufacturers normal quality control program, the test shall follow steps 1 to 4 of the loading procedure given in Part 5 of Reference 113.

17.12.1 Erection Tolerances

Figure 63 illustrates many of the erection tolerance requirements.

Figure 62

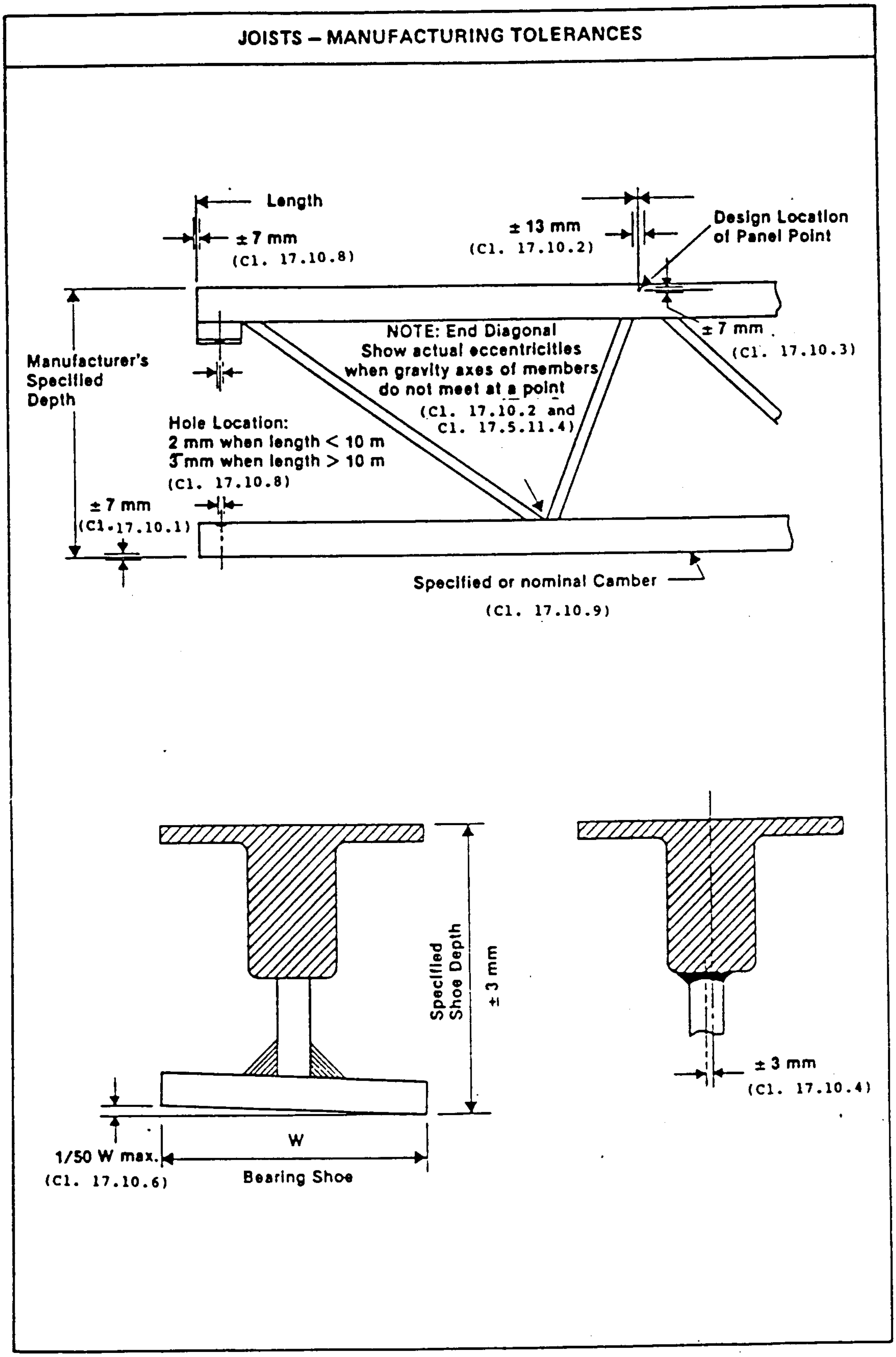
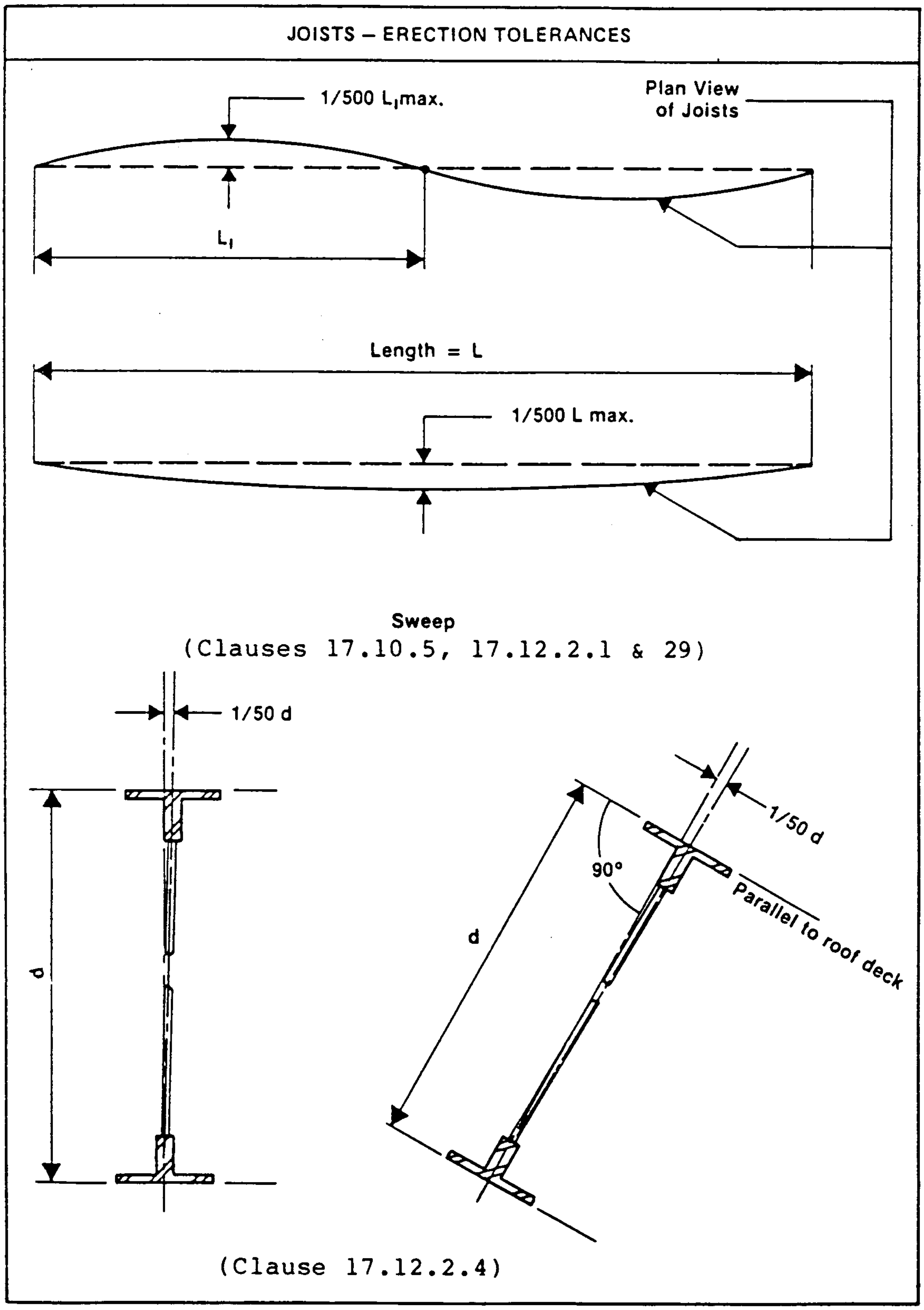

JOISTS – MANUFACTURING TOLERANCES
Length
± 7 mm
(Cl. 17.10.8)
± 13 mm
(Cl. 17.10.2)
Design Location
of Panel Point
± 7 mm
(Cl. 17.10.3)
Manufacturer's
Specified
Depth
NOTE: End Diagonal
Show actual eccentricities
when gravity axes of members
do not meet at a point
(Cl. 17.10.2 and
Cl. 17.5.11.4)
Hole Location:
2 mm when length < 10 m
3 mm when length > 10 m
(Cl. 17.10.8)
± 7 mm
(Cl. 17.10.1)
Specified or nominal Camber
(Cl. 17.10.9)
Specified
Shoe Depth
± 3 mm
± 3 mm
(Cl. 17.10.4)
W
1/50 W max.
(Cl. 17.10.6)
Bearing Shoe

Figure 63


JOISTS – ERECTION TOLERANCES
1/500 L_1max.
Plan View of Joists
L_1
Length = L
1/500 L max.
Sweep
(Clauses 17.10.5, 17.12.2.1 & 29)
1/50 d
d
1/50 d
90°
d
Parallel to roof deck
(Clause 17.12.2.4)

18. COMPOSITE BEAMS AND COLUMNS

Clause 18 provides specific requirements for the design of:

(a) Composite beams consisting of steel sections, trusses or joists interconnected by means of shear connectors with either a reinforced concrete slab or a steel deck and concrete cover slab;

(b) Composite beams consisting of steel sections or joists fully encased in concrete interacting with a concrete slab;

(c) Composite columns consisting of hollow structural steel sections filled with concrete.

18.2 Definitions

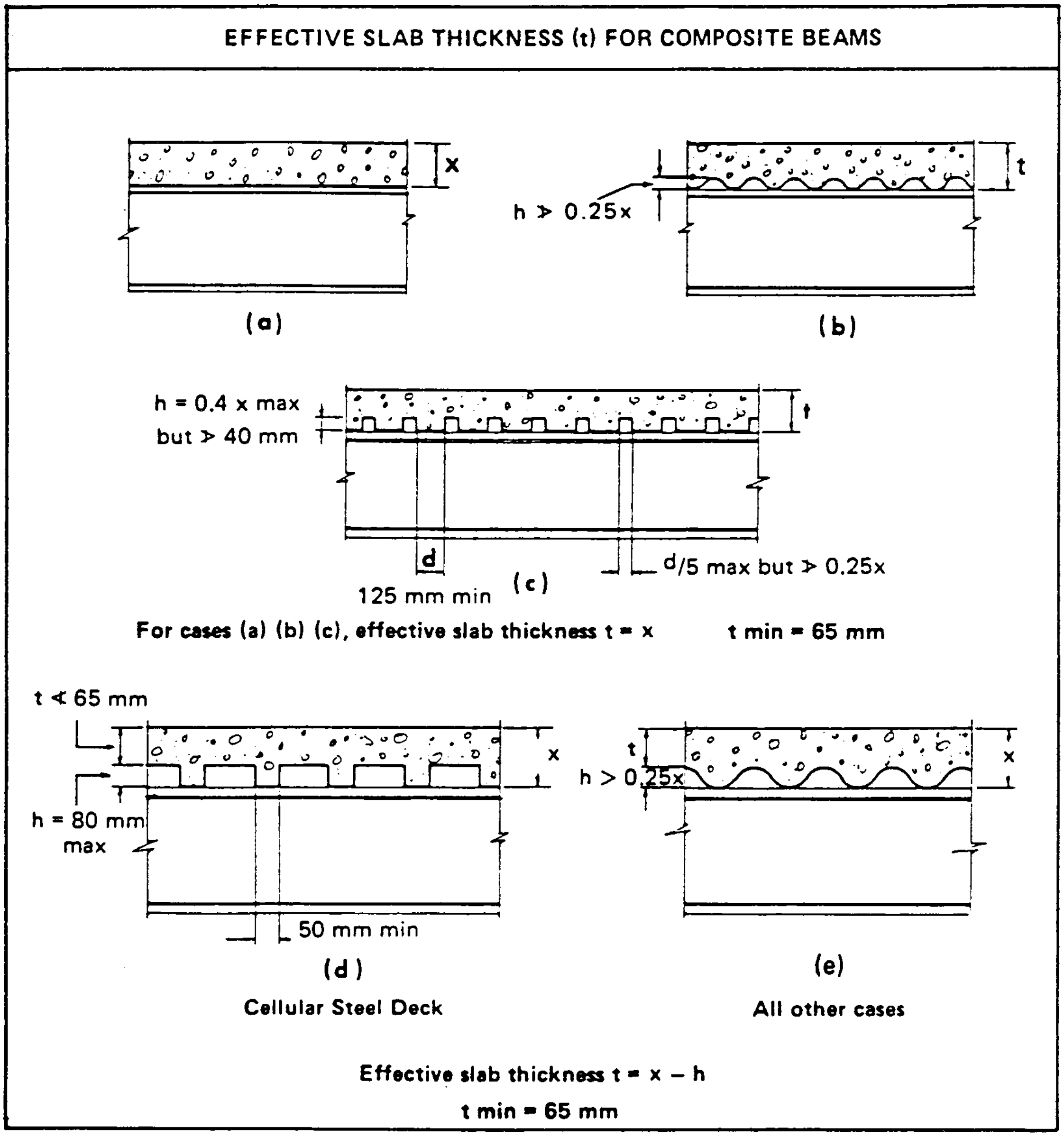
Figure 64 illustrates various cases of effective slab and cover slab thickness.

18.3 Composite Beams

18.3.1 General

The arbitrary increases to the elastically computed deflections for the effects of creep of concrete, partial shear connections and from slip of the steel-concrete interface affords a convenient means to check deflection against any prescribed

Figure 64



deflection limit.[126,127,128,129,141] The deflection due to concrete creep is of concern for "permanent" loads (e.g. dead loads and live loads of long duration). The equation given in the Standard for the effective moment of inertia when increased flexibility results due to partial shear connection is based on a re-evaluation of the data given in 126. These data also show that the slip at the concrete-steel interface, even with 100 per cent shear connection, causes a 15 per cent increase in deflections.

Member deflection resulting from shrinkage of the concrete depends of course on all those factors affecting the concrete shrinkage such as the age of concrete when loaded, ratio of slab volume to surface area and concrete properties (slump, % fines, entrained air and cement content). Careful construction practices such as the provision of proper moist curing conditions will minimize shrinkage strains. For structures of usual proportions and with usual construction practices deflections of a simply supported composite beam due to shrinkage strains have been shown by calculation to be about equal to that of the dead load.

Because the concrete is not assumed to carry any vertical shear the steel member must carry the total vertical shear. Similarly, the end connections must carry the total end reaction of the composite beam.

18.3.2 Design Effective Width of Concrete

The width of slab or cover slab deemed to be effective, when computing the concrete force, is the least of the three values given in the clause. These rules were formulated on the basis of elastic conditions (see, for example, Ref. 75). Reference 111 suggests that conditions at ultimate load are somewhat different. However analyses show that changes in moment capacity due to differences in design effective widths are negligible.

18.3.3 Slab Reinforcement

The longitudinal shear forces generated by interconnecting concrete slabs to steel sections or joists by means of shear connectors sometimes cause longitudinal cracking of the slab directly over the steel. This is independent of any flexural cracking which may occur due to the slab spanning continuously over supports, although the two effects may combine. Longitudinal shear cracking is more apt to start from the underside of the slab whereas flexural cracking is more apt to start at the top surface of the slab. Some investigation has been carried out[76,77] but a full explanation and corresponding design procedure is, to date, not available.

This phenomenon has not been observed in cover slabs over steel deck, possibly because the steel deck provides a measure of reinforcement. Reinforcement of the cover slab for other reasons, as may be indicated by consideration of flexure, fire resistance, shrinkage or temperature effects may, however, be necessary.

18.3.5 Interconnection

Except in the case of unpainted sections or joists supporting slabs, and totally encased in concrete to a depth of 50 mm on sides and soffit, shear connectors are required to effect the interconnection of steel and concrete. While studs are assumed to be the usual method of interconnection and are the subject of most of the specific requirements in subsequent clauses, other means of interconnection which have been shown to be satisfactory may be used instead. Where end-welded steel studs are welded through steel deck both the total sheet thickness and the total amount of zinc coating are limited in accordance with successful field experience to date.

Tests have shown that a shear connector is not fully effective if welded to a support which is too thin or flexible[85]. For this reason the stud diameter is limited to 2.5 times the thickness of the part to which it is welded unless a lesser thickness can be justified.

18.3.6 Shear Connectors

The general intent of this clause is to require the use of shear connector resistances, which have been satisfactorily established by test. Certain types of shear connectors (studs, channels) have been extensively tested in the past and therefore permissible values are given which are considered acceptable without further verification.

In 2.7A - Limit States Design a resistance factor $\phi_{sc} = 0.80$ is applied to shear connector resistances to modify the nominal member or connector resistances to account for possible variations in material strength, dimensions of the part, and so on.

The values given in Clause 18.3.6(a) are based on an extensive research program[78] involving both normal and lightweight concrete slabs. The limiting value of $\phi_{sc}\ F_u\ A_{sc}$ in LSD or $F_u\ A_{sc}$ in WSD represents the tensile strength of the stud times its cross-sectional area (A_{sc}) for, as the concrete pushes against the stud, the stud eventually begins to bend over and develop a tensile resistance. The same values may be used in designs incorporating studs passing through the flutes of steel deck into the concrete cover slab above provided that the flute average width is at least twice its height and the stud extends into the cover slab at least 2 diameters[79].

Based on work presented in Reference 75, numerical values are given in Table 8 for studs used in conjunction with steel deck in which the flute average width is less than twice its height. It was also found that the value of a pair of studs in a flute was less than twice the value of a single stud and the values given reflect these findings. A similar reduction does not appear to be needed for pairs of studs in solid slabs; in this case the value of a pair is simply assumed to be twice the value of a single stud. It is recommended, however, that where studs are used in pairs the lateral spacing between them be not less than 4 stud diameters measured centre to centre of studs in order to minimize excessive localized stress in the surrounding concrete.

The value assigned to channel connectors[80] in LSD is the same as used in WSD except that it is in a different format as in both cases the connectors are designed for the ultimate shear on the interface.

18.4 Design of Composite Beams with Shear Connections - Limit States Design

The factored moment resistance of a composite beam is computed based on the ultimate capacity of the cross-section[26,29,75,81,82,83,84] where the following assumptions are made:

(1) Concrete in tension is neglected.

(2) If a steel joist is used, only the lower chord is considered effective when computing the moment resistance.

(3) Tension and compression forces are in equilibrium about the plastic neutral axis.

(4) Forces are obtained as the product of a limit states stress (ϕF_y for steel and $0.85\ \phi_c f'_c$ for concrete) times the appropriate area.

(5) The resistance factor for concrete (ϕ_c) is set at a lower value (0.60) than that used for steel members (0.90) to take into account the greater variability in the strength and dimensions of concrete elements.

Three design cases, 1,2 and 3, are considered where:

(1) Case 1 represents full shear connection and the plastic neutral axis in the slab.

(2) Case 2 represents full shear connection and the plastic neutral axis in the steel section.

(3) Case 3 represents partial shear connection for which the plastic neutral axis is always in the steel section.

When joists are used, only Case 1 is permitted since this provides for an efficient design and a predictable mode of failure. For Case 3, the position of the compressive resistance of the concrete is determined by the expression for "a" as suggested in Reference 84.

18.4.4 (LSD) Where partial shear connection is used, a lower limit of 50 percent of the lesser of 0.85 $\phi_c f'_c bt$ and $\phi A_s F_y$ is imposed for flexural strength considerations. The limit is reduced to 25 percent for deflection considerations. The lower limit for strength evaluation is based on studies which indicate that, at some value below one half the number of shear connectors required for full shear connection, the "flexibility" at the plane of interconnection has increased to the point where the basic assumptions as to an integral composite section may not apply over the whole loading range. The lower limit for deflection design is less stringent because deflection is normally of concern up to the level of specified load rather than the higher level of factored load on which strength design is based.

18.4.5 (LSD) If the factored resistance provided by the shear connectors is at least equal to the factored horizontal shear force $\phi A_s F_y$, then full shear connection has been provided for Case 1 (plastic neutral axis in the slab). For full shear connection for Case 2 (plastic neutral axis in the steel), the factored resistance provided by the shear connectors must be at least equal to 0.85 $\phi_c btf'_c$. These values are obtained by considering the forces above or below the plane on which the horizontal force acts, that is, the interface between the steel and the concrete. Since conditions at the location $M = M_{max}$ are implicitly being considered, this total horizontal shear force is the summation of connector resistances between that location and any adjacent point of $M = O$.

18.4.8 (LSD) Shear connectors may be spaced uniformly in most cases. An exception is the case where, in a region of positive bending, a concentrated load or loads occurs between the point of zero moment and the point of maximum positive moment. This clause ensures that the moment capacity of the composite section at the point of concentrated load is achieved. If n represents the total number of shear connectors to develop the maximum moment, M_f, and n', the number to develop the moment M_{f_1} at the point of concentrated loads, then it might be assumed that $n'/n = M_{f_1}/M_f$. However the steel section alone has a moment capacity, M_r, which is not dependent on the presence or absence of shear connectors. This capacity is subtracted from both M_f and M_{f_1} to obtain the equation given in Clause 17.4.8.

18.4 Design of Composite Beams with Shear Connectors - Working Stess Design

18.4.6 (WSD) The determination of the required number of shear connectors for complete composite action with concrete in flexural compression is based on the assumption that at ultimate load the entire horizontal shear acting at the junction of the steel beam and the concrete slab is transferred by shear connectors alone. This horizontal shear (V_h) cannot be greater than either A_sF_y or $0.85\ f'_cA_c$ so that the lesser value represents the horizontal shear force at the limit of flexural usefulness of the composite beam. A_c is the area of concrete equal to effective flange width times effective slab thickness. From V_h thus determined the number of shear connectors N_u required between a point of maximum positive moment and an adjacent point of zero moment is $n = V_h/q$ where q is the load per connector given in Table 13 of the Standard.

18.4.7 (WSD) In the negative moment regions of continuous composite beams advantage may be taken of suitably anchored longitudinal reinforcement in the slab when computing section properties. In this case the reinforcement must be anchored into concrete which is in compression and sufficient shear connectors must be provided between the point of maximum negative moment and an adjacent point of zero moment to resist a horizontal shear $V_h = A_{sr}F_{yr}$. Research to date is not entirely definitive as to the value of shear connectors in regions of negative bending but reported results indicate that the reinforcement can be considered effective if properly anchored and the number of shear connectors as indicated above is provided.

18.4.8 (WSD) Shear connectors may be spaced uniformly in most cases. An exception is the case where, in a region of positive bending, a concentrated load or loads occurs between the point of zero moment and the point of maximum positive moment. This clause ensures that the moment capacity of the composite section at the point of concentrated load is achieved. If n represents the total number of shear connectors to develop the full, M_{max}, and n' the number to develop the moment, M, at the concentrated load, then it might be assumed that $n'/n = M/M_{max}$. However, the steel section alone has a moment capacity, M_{steel}, which is not dependent on the presence or absence of shear connectors. This capacity is substracted from both M and M_{max} to obtain the equation given in Clause 17.4.8.

18.5 Design of Composite Beams Without Shear Connectors

As a conservative simplification Clause 18.5.3 permits the design of encased simple span steel sections or joists as if the steel section or joist alone supported 90 percent of the total load. This is a tacit assumption that the composite section containing the encased steel section or joist will have a moment resistance at least 11 percent greater than the bare steel member. Typically, the moment resistance computed according to Clause 18.5.2 would show a larger increase over the bare steel member.

18.6 Unshored Beams

To keep strains resulting from the application of specified load within the elastic range it is required to limit the stress (as a measure of strain) in the tension flange of the steel section or joist to 0.90 F_y under specified load. The state of stress or strain of the bottom fibre of steel at specified loads has no effect whatsoever on the ability of the composite beam to reach its ultimate moment. This clause serves to guard against permanent deformations under specified loads.

18.8 Design of Composite Columns

Where hollow structural sections (HSS) used as columns are completely filled with concrete, advantage may be taken of the increased load capacity, which will be highest for stocky members and minimal for slender members. Many research programmes have demonstrated that composite columns of this type have improved load-carrying ability within a certain range of parameters[87,130,131,132,133].

Generally the axial load to be carried by the concrete should be applied by direct bearing on the concrete although recent experiments[134] show for relatively long columns, especially when loaded eccentrically, that bond and interlocking due to the curvature developed may suffice. For axially loaded columns the method of superposition proposed in Reference 132, where the total load is taken as that carried by the steel plus that carried by the concrete is used. Each contribution is a function of the slenderness ratio of that part. For the concrete, the creep of the concrete due to long term loading is taken into account as is the increase in axial load capacity of the concrete due to the confining pressure exerted by circular hollow sections.

For combined axial compression and bending it is assumed conservatively that all bending moments are resisted by the steel which also is assumed to carry a net axial load equal to the total axial load less that sustained by the concrete.

19. GENERAL REQUIREMENTS FOR BUILT-UP MEMBERS

The term built-up member refers to any structural member assembled from two or more components. Such members may be used to resist compression, tension or bending and the requirements for fastening together the various components vary accordingly.

In compression members it is necessary to prevent local buckling of components at loads less than those which would cause the member to buckle as a whole. In tension members, buckling is not a consideration but it is generally desirable that components be stitched together sufficiently to work in unison and to minimize vibration. With exposed members it is also important that components in contact be fitted together tightly enough to minimize possible corrosion problems. The sketches comprising Figure 65 illustrate the main provisions of Clause 19.

20. STABILILTY OF STRUCTURES AND INDIVIDUAL MEMBERS

20.1 General

There is an emphasis in this Standard on the designer's responsibility to ensure stability of the structure and the individual members (see also Commentary on Clause 8.6); on the interconnection of various load resisting elements, for example, use of floors or roofs as diaphragms; on proper load transfer, for example, use of one bent to provide resistance to lateral load for several, more flexible, adjacent bents or the use of end walls for lateral load resistance for the central structure, and on the necessity to show information on design and erection drawings.

20.2 Stability of Columns

The clause relates to a local requirement and reminds the designer that the beam-to-column connection may have to be capable of resisting a tensile or compressive component as well as the usual shear or moment. By basing the force requirements on the anticipated out-of-plumb coupled with the flexibility of the structure, a less conservative approach to the problem is anticipated. (See also the commentary on Clause 8.6)

20.3 Stability of Beams, Girders and Trusses

The requirement of Clause 19.3.1 is based on the work reported in References 89 and 90. For members in plastically designed structures under uniform moment conditions the force may be higher than that specified[91].

Figure 65(a)

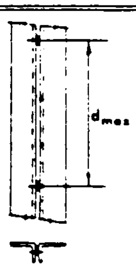
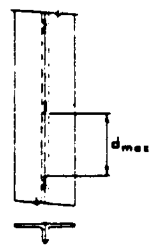
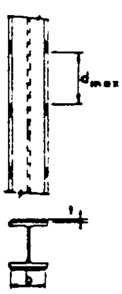
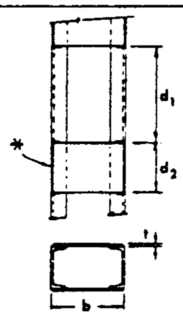
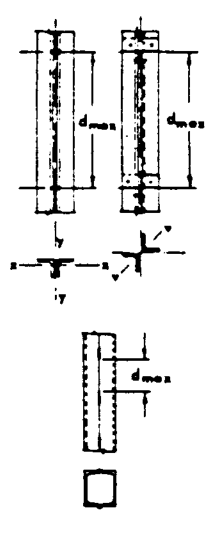
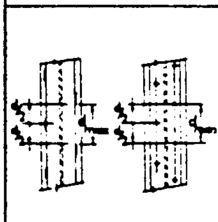
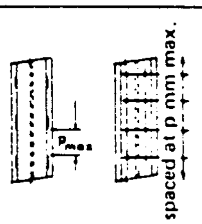
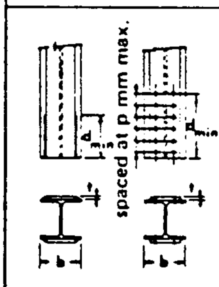
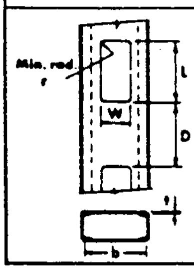
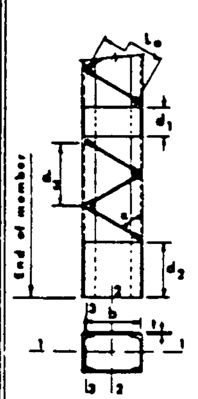
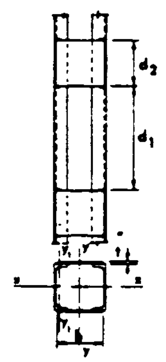
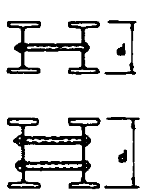
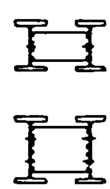
BUILT-UP MEMBER DETAILS			
Tension Members	Requirements	Tension Members	Requirements
	TWO ROLLED SHAPES NOT IN CONTACT d_{max} = 300 × least radius of gyration of one component		TWO ROLLED SHAPE IN CONTACT d_{max} = 600 mm d_{max} may be increased when justified.
	SHAPE AND PLATE IN CONTACT d_{max} = 36 t or 450 mm whichever is lesser.		BATTENS $b < 60t$ $d_2 > \frac{2}{3}b$ $d_1 < 300$ × least radius of gyration of one component. *For intermittent welds or fasteners max pitch = 150 mm

Figure 65(b)

BUILT-UP MEMBER DETAILS

Compression Members	Requirements
	ROLLED SHAPES $d_{max} = \left(\frac{KL}{r}\right) r_{min}$ $\frac{KL}{r}$ = Design slenderness ratio of member as a whole. r_{min} = Least radius of gyration of one component 19.1.3
	STAGGERED FASTENERS OR WELDS $d_{max} = \frac{525}{\sqrt{F_y}}$ or 450 mm t = Outside plate thickness 19.1.3
	FASTENERS OR WELDS NOT STAGGERED $P_{max} = \frac{330}{\sqrt{F_y}}$ or 300 mm t = Outside plate thickness 19.1.3
	ENDS OF BUILT-UP COLUMNS Welded Connections. $d_{min} = b$ Bolted Connections: $d_{min} = 1.5\ b$ P_{max} = 4 x diameter of fastener 19.1.2
	$b < \frac{840}{\sqrt{F_y}}$ Note: For bolted fabrication b > 400 mm is preferred. $L < 2W$ PERFORATED COVER PLATES $D > b$ $r > 40$ mm 19.1.2

Compression Members	Requirements
	LACING AND TIE PLATES $b < 60t$ $d_1 > \frac{b}{2}$ $d_2 > b$ $d_3 < \frac{KL}{r_1} r_3 < \frac{KL}{r_2} r_3$ $L_e < 140 \times$ rad. of gyration of lacing member. $\alpha \geq 45°$ KL = Effective length of member referred to appropriate axis 19.1.4.9
	BATTENS $b < 60t$ $d_2 > b$ When $\frac{K_y L_y}{r_y} < 0.8 \frac{K_x L_x}{r_x}$ $\frac{d_1}{r_{y_1}} < 50 \quad < 0.7 \frac{K_x L_x}{r_x}$ When $\frac{K_y L_y}{r_y} > 0.8 \frac{K_x L_x}{r_x}$ $\frac{d_1}{r_{y_1}} < 40 \quad < 0.6 \frac{K_y L_y}{r_y}$ 19.1.13

Beams and Grillages	Requirements
	For non-load-sharing beams only, through bolts and separators may be used. 19.3 Not less than: One bolt —— d < 300 mm Two or more bolts – d > 300 mm Centres of separator groups < 1 500 mm
	Diaphragms shall be used for sharing loads. 19.3 Diaphragms shall have sufficient stiffness to distribute required load. Centres of diaphragms < 1 500 mm

When an element in a structure must resist the "bracing forces" from more than one member it is not possible to state explicitly how these forces should be combined, since the out-of-straightness giving rise to the bracing forces may vary both in magnitude and sense. The conservative assumption would be that all forces act in the same sense at the maximum magnitude. Reference 109 provides more guidance in selected cases.

Because the shear centre of an asymmetric section does not coincide with the centroid, this section may be unintentionally loaded so as to produce torsion and biaxial bending. Both the connections and the members providing reactions should be checked.

21. CONNECTIONS

21.3 Restrained Members

When the compressive or tensile force, transmitted by a beam flange to a column (for LSD approximately equal to the factored moment divided by the depth of the beam and for WSD equal to the specified yield point of the beam flange multiplied by the area of the flange) exceeds the bearing or tensile resistance of the web of the column, stiffeners are required to develop the load in excess of the bearing or tensile resistance.

Reference 60 recommends that the length of the column web risisting the compressive force be taken as the thickness of the beam flange plus 5k. This length multiplied by the thickness of the column web and the specified minimum yield point of the column gives the bearing resistance for the column web for Class 1 and 2 webs.

The same reference, using conservative assumptions as to the relative dimensions of beams and columns, computes the column flange bending resistance when subjected to tensile load from the beam flange, to be 7 $t_c^2 F_{yc}$. Tests have shown that connections proportioned in accordance with this equation have carried the plastic moment of the beam satisfactorily.

For members with Class 3 and 4 webs, the bearing resistance of the web is limited by its buckling strength. The expression for bearing resistance is conservatively based on the critical buckling stress for a plate with both edges simply supported.

$$\sigma_{cr} = \frac{\pi^2 E}{12(1-\nu^2)(h/w)^2} \qquad k = \frac{723\ 000}{(h_c/w_c)^2}, \text{ when } k = k_{min} = 4$$

The coefficient 640 000 given in the LSD Clause 21.3(a) for the factored bearing resistance reflects a further reduction of about 10 percent to account for uncertainty in the prediction of the member strength and thereby to maintain a more uniform level of probability against failure . The coefficient 380 000 given in WSD Clause 21.3(a) for the bearing resistance reflects the same reduction of about 10 percent but also includes a factor of safety of 1.67.

21.4 Connections of Tension or Compression Members

The 50 percent rule provides for a reasonably sized connection, especially when a member has been designed for stiffness rather than strength.

21.5 Lamellar Tearing

This clause has been included in recognition of the widespread use of welded joints in steel structures for buildings. In cases where shrinkage results as a consequence of welding under highly restrained conditions, very large tensile strains may be set up. If these are transferred across the through thickness direction of rolled structural members or plates, lamellar tearing may result. If this type of joint cannot be avoided, steps may be taken to minimize the possibility of lamellar tearing[93,94].

21.6 Placement of Fasteners and Welds

Reference 95 has shown that, except for cases of repeated loads, end welds on tension angles and other similar members need not be placed so as to balance the forces about the neutral axis of the member.

21.8 Fillers

In bearing-type shear connections development of the filler before the splice material diminishes bending of the bolt. In slip-resistant joints, tests with fillers up to 1 inch in thickness and with surface conditions comparable to other joint components show that the fillers become an integral part of the joint and they need not be developed before the splice material[55].

22. BOLTING DETAILS

While Clause 13.9(c) specifies the bearing capacity of the plate material as a function of end distance Clause 22.8 specifies the minimum edge distances permitted for connections of tension members with one, two or more bolts for various bolt diameters. Reference 55 contains a comprehensive summary of bearing strengths as a function of end distance and the types of failures that are guarded against.

23. STRUCTURAL JOINTS USING ASTM A325 OR A 490 BOLTS

23.1 General

A325M[137] A490M[138] A325[97] and A490[98] bolts are produced by quenching and tempering. A325 bolts are somewhat lower in tensile strength than A490 bolts but have greater ductility. These types of bolts are intended to be initially tensioned to at least 70 percent of their specified minimum tensile strengths and, therefore, exert a high clamping force on the parts which they join. When joints are required to resist shear between connected parts, it is required that the design and shop drawings specify the joints as either slip-resistant or bearing-type. As explained in the Commentary on Clause 13.11, the shear transfer in slip-resistant joints results from the action of the clamping forces in the bolts upon the faying surfaces. This shear transfer is to be examined at specified load levels, and it is expected that this type of connection will be used primarily in connections subjected to load reversal or where slip into bearing would result in unacceptable geometry changes in the structure.

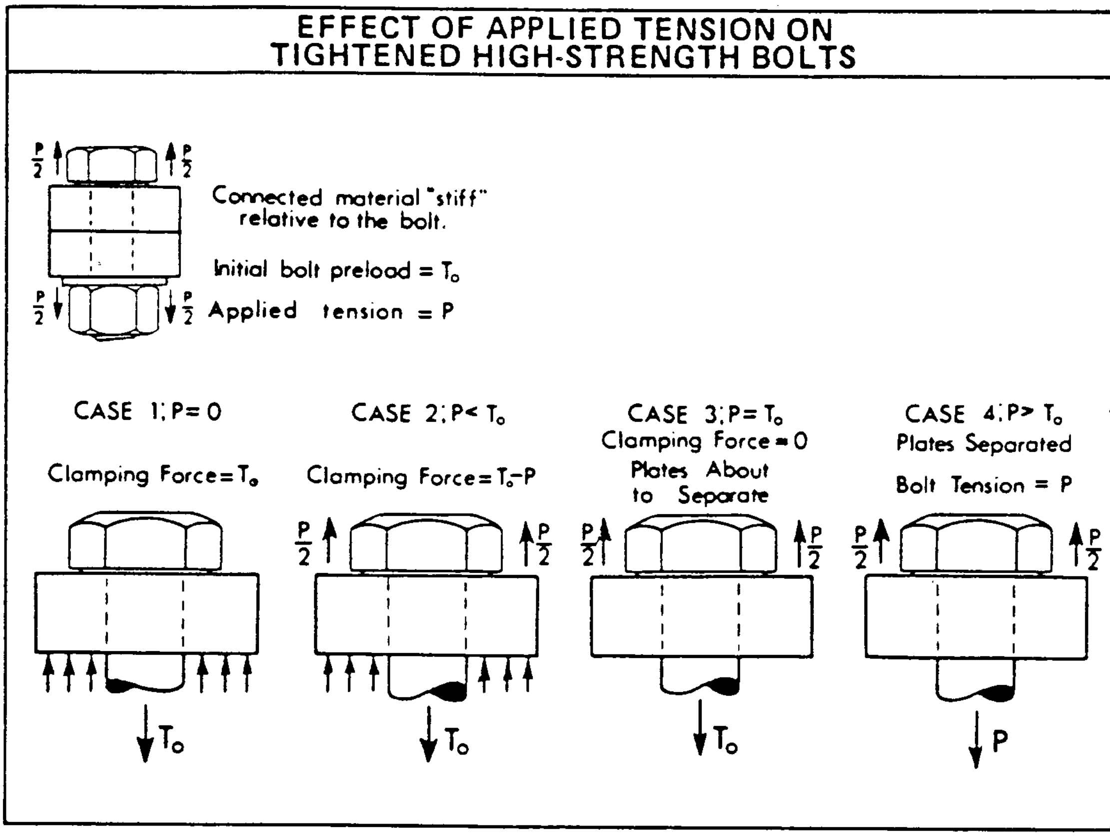
Although the load transfer in bearing-type connections will also be by friction at low load levels, slip into bearing will occur well before factored load levels (LSD) or specified load levels (WSD) are reached. Thus the controlling strength elements are shear of the bolt across the shank of the bolt (Clause 13.10) and the bearing capacity of the plate adjacent to the bolts (Clause 13.9(c)). Since there are few joints in building construction that are subject to load reversal nor are there many situations where a one-time slip into bearing cannot be tolerated, bearing-type joints would be the usual choice.

As a result of normal fabrication practice, minor misalignment of bolt holes may occur in connections with two or more bolts except when all parts of the connection are match punched or drilled. Such misalignment, if anything, has a beneficial effect[55] in offering a stiffer joint, improved slip resistance and decreased rigid body motion.

23.1.5 Applied Tension

Figure 66 illustrates that, when an external tensile force is applied to the connected parts, the applied external force and the internal clamping force in the bolt are not additive. The bolt needs to be proportioned only for the tensile load. The illustration (Fig. 66) assumes 'stiff' connected material relative to the bolt. Measurement of actual bolt forces in connections of practical sizes has shown that there is an

Figure 66

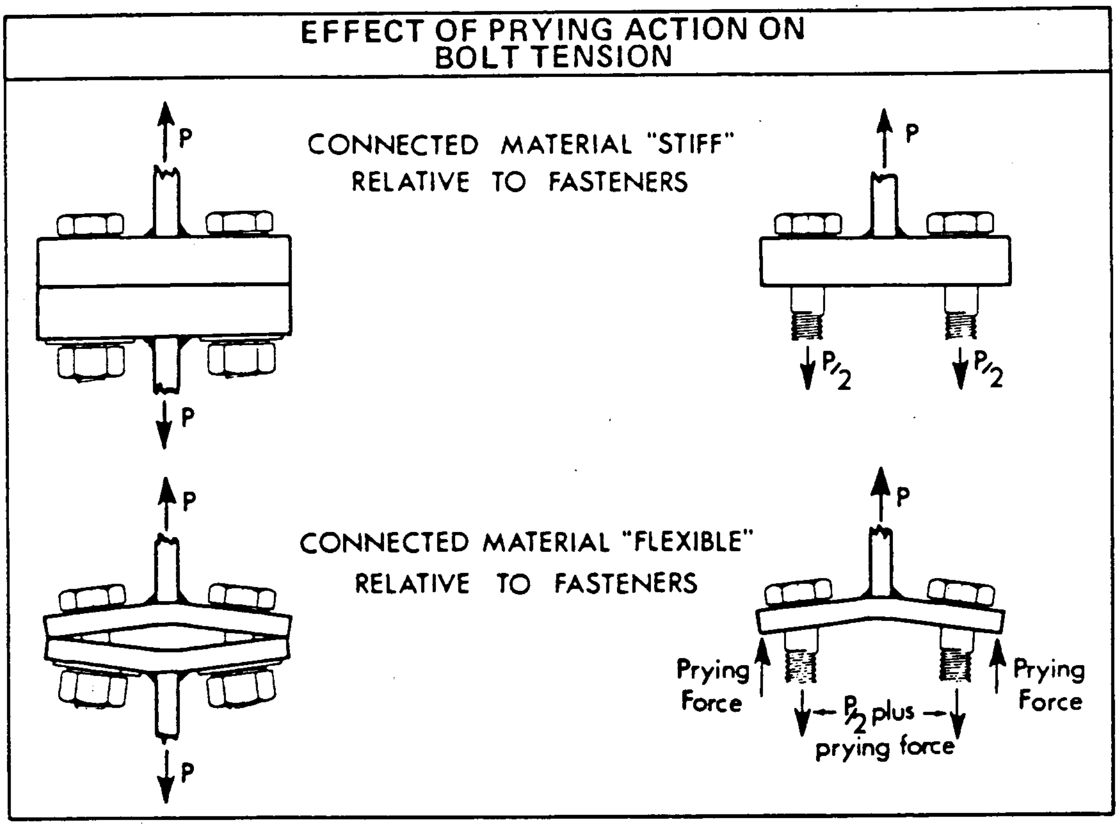


increase in bolt force over that assumed but it is modest, usually about 5-10 percent.

The effect of prying action on bolt tension, illustrated in Figure 67, is an important design consideration. Page 278 of the "Guide to Design Criteria for Bolted and Riveted Joints" summarizes the criteria for the design of tension connections when prying action is present. For load reversal or repeated load situations these connections must be proportioned so that prying is avoided.

The Research Council on Structural Connections is currently (1980) considering specific limitations on the allowable tension

Figure 67



for different ranges of stress cycles provided the prying forces are less than 60 percent of the external load. These limits are based on an evaluation of available test data. Until more guidance is available designers should consider a conservative design approach to these connections.

23.2 Bolts, Nuts and Washers

The "standard" A325M, A490M, A325 or A490 bolt has a heavy hex head, restricted thread length, coarse threads and is equipped with a heavy hex nut. Proprietary versions are available which differ from the "standard" type in various aspects, and in some cases may offer one or more advantages.

Their use is permissible under the conditions set forth in Clause 23.2.4.

Galvanized A325M and A325 bolts are permitted by Clause 23.2.3; however galvanized A490 bolts are not permitted since they are especially susceptible to stress corrosion and hydrogen stress cracking.[55]

23.3 Bolted Parts

As permitted by Clause 23.3.4, the faying surfaces of slip-resistant joints may be hot-dip galvanized or coated with sprayed metal coatings, zinc-rich paints, or a vinyl surface treatment. Other materials and methods may also be used when approved.

23.4 Installation

Even though the Standard divides the design of shear joints into bearing-type and slip-resistant connections, the installation procedure is the same for all bolts. As has already been described, the high initial bolt tension is necessary for slip-resistant joints. Although the ultimate shear strength of a high strength bolt is independent of the initial clamping force[55], the presence of such a force provides a better stress pattern and security against nut loosening for either type of connection. In addition, the use of a consistent procedure for all bolted joints minimizes the potential for errors.

Except when galvanized, A325M and A325 bolts may be reused once or twice, providing that proper control on the number of reuses can be established55,96. A490 bolts should not be reused.

23.5 "Turn-of-Nut" Tightening

Any installation procedure used for high strength bolts involves elongating the bolt so as to produce the desired tensile force. Although part of the bolt probably remains elastic (the shank), the threaded portion behaves plastically. It is because the bolt as a whole is tightened into the inelastic range (out onto the flat portion of its load vs. deformation response curve), that the exact location of "snug-tight" is not critical. For the same reason, application of the specified amount of nut rotation will result in preloads which are above those prescribed in Table 15 and which are not greatly variable. There is a reasonable margin against twist-off but the tolerance on nut rotation prescribed in the footnote to Table 16 is good practice, particularly when galvanized A325 bolts or black A490 bolts are used.

23.6 Tightening by Use of a Direct Tension Indicator

The Standard permits use of direct tension indicator bolting systems. All of these are proprietary in nature but all rely on some physical change in some part of the bolt system to indicate when the minimum bolt tension has been achieved. For example, one such system relies on a physical gap being closed down to a specified dimension which can be measured with the appropriate tool.

23.7 Inspection

Bolts, nuts and washers are normally received with a light residual coating of oil. This coating is not detrimental, in face it is desirable, and should not be removed. Galvanized bolts and/or nuts may be coated with a special lubricant to facilitate tightening. Obviously, this should not be removed.

Bolts tightened by the turn-of-nut method may have the outer face of the nut match-marked with the bolt point before final tightening, thus affording the inspector visual means of noting nut rotation. Such marks can be made with crayon or paint by the wrench operator after the bolts have been snugged.

The sides of nuts or bolt heads tightened with an impact wrench appear slightly peened and thus indicate that the wrench has been applied. For bolts in a bearing-type connection subjected only to shear, this evidence that the nuts have been tightened is sufficient inspection since the ultimate shear strength of the bolt is independent of the amount of preload.

When a torque wrench is used in inspection to verify bolt tension, the procedures to be followed are described in detail in Clause 23.7.4. A washer under the turned element is necessary to minimize erratic torque-tension relationships.

25. COLUMN BASES

In general the use of base plates bearing directly on grout is preferred to the use of levelling plates interposed between the base plate and the grout. The latter condition may lead to uneven bearing.

When the supporting surface is wider on all sides than the loaded area the compressive resistance of the concrete is increased because of triaxial effects. When compression exists over the entire base plate area rather than using a linearly varying pressure it is reasonable to use a uniform pressure on a depth equal to d-2e because it is the average pressure which would result in failure of the concrete rather than the maximum value at the edge of the base plate.[183]

Resistances of the concrete to pull-out and transverse loads are in accordance with CAN3-A23.3, Design of Concrete Structures for Buildings[184]. Tests have shown[136], that when shear is transmitted by bearing of the anchor bolts on the concrete, the bearing area can be taken as the product of the bolt diameter and an assumed depth of 5 times the bolt diameter.

26. ANCHOR BOLTS

Anchor bolts may be required to resist the effects of uplift forces, bending moments and shears applied to the anchorage. The tensile resistance is determined on the basis of the ultimate tensile strength using the tensile stress area of the bolt as the net area. This area when National Coarse threads are used is[139]:

$$A_n = \frac{\pi}{4} (D - 0.97p)^2$$

The shear resistance of bolts is based on the expression given in Clause 13.10.2. For combined shear and tension an elliptical interaction equation is used as given in Clause 13.10.4. Tests have shown[185], when anchor bolts are subjected to combined shear and tension, that the grout breaks away and does not support the anchor bolt. The bolts are therefore subjected to bending and depending on the fixity or lack of it at the base plate, may be bent in single or double curvature. The bolts must be designed against combined tension and bending. The bending strength is based on the elastic section modulus rather than the plastic section modulus because some steels used for anchor bolts have limited ductility.

31. PLASTIC DESIGN - WSD

31.1 (WSD) General. Plastic design, based on the plastic theory is a method of proportioning steel structures on the basis of maximum useable strength rather than on allowable stress. In general, it is assumed that a plastically designed frame achieves its limit of usefulness when sufficient plastic hinges have developed to tranform the frame into a mechanism. The plastic method relies on certain basic assumptions for its validity[12] and this WSD Standard imposes the necessary restrictions in order to preserve the applicability of the plastic theory. The basic restriction pertains to the steel itself and is contained in Clause 8.5(a) which in effect states that the steel specified shall be characterized by a plateau in the stress-strain curve at the yield stress level and exhibit strain-hardening when the average strain exceeds the plastic strain. This behaviour should be evidenced at the temperatures to which the structure will be subjected in service. Also, although not stated, plastically designed structures usually entail welded fabrication, so the steel specified should also be weldable.

31.2 (WSD) Permissible Types of Members and Frames. Plastic design has been extended to all members in structures prevented from sidesway by bracing, shear walls or other effective means[19,20]. Thus braced multi-storey buildings of any height may now be proportioned in accordance with Clause 31. In addition unbraced rigid frames not exceeding two storeys in height also may be proportioned in accordance with Clause 31.

31.2 (WSD) Load Factor. In general, the factored load P is taken as:

$$P = \alpha_D D + \psi \gamma \alpha_L (L + Q + T)$$

where ψ = load combination factor given in Clause 7

α_D = load factor of 1.4

α_L = load factor of 1.7

γ = importance factor

31.4 (WSD) Tension Members. In order to base the design of tension members on the same criteria as used in allowable stress design the tensile load is limited to $1.67 A_n F_t$. In general F_t = $0.6\ F_y$ but in certain cases, as per Clause 13.2, $F_t < 0.6\ F_y$. In addition, in order to ensure that yielding precedes fracture, it is specified that the ratio A_n/A_g shall be greater than F_y/F_u.

31.5 (WSD) Compression Members. The compressive load is limited to $1.67\ A_g F_a$ (i.e., 1.67 times the compressive load permitted by allowable stress design).

31.6 (WSD) Beam-Columns. Interaction equations of the same type as given in Clause 13.7, except in terms of load and moment, rather than stress, are used to proportion beam-columns. However, other published and recognized methods of beam-column analysis may be used if acceptable to the Regulatory Authority. Because the given interaction equations of necessity are generalized, other means of analysis may be appropriate within a more limited range of parameters.

31.7 (WSD) Web Crippling. Web stiffeners are required on a member at a point of load application where a plastic hinge would form. Stiffeners are also required at beam-to-column connections where the loads delivered by beam flanges would either cripple the column web or, in the case of tension loads, curl the column flange. The rules for stiffeners are essentially the same as those given in Clause 21.3.

In lieu of pairs of stiffener plates parallel to and approximately in line with the flanges of the member delivering the load, plates parallel to the column web and attached to the toes of the column flanges may be used. Reference should be made to the technical literature[12] for further details of stiffeners and for special requirements pertaining to tapered and curved haunches[16].

31.8 (WSD) Width-Thickness Ratios. In order to preclude local buckling and to ensure adequate hinge rotation, compression elements in regions of plastic moment must have width-thickness ratios no greater than those specified for Class 1 sections in

Clause 11.3. Sections which meet such requirements are termed "plastic design sections" to differentiate them from the Class 2 and Class 3 sections called for in allowable stress design. Plastic design sections are more restricted in width-thickness ratios than are Class 2 sections. The former must be able to withstand substantial rotation at a plastic hinge locations whereas the latter need have only limited rotation capacity.

31.9 (WSD) Connections. This requirement is to ensure that splices located at inflection points have sufficient capacity to enable the structure to act as if continuous up to the ultimate load.

APPENDIX L - COLUMNS SUBJECT TO BIAXIAL BENDING (LSD)

These refined expressions were introduced in an Appendix in order not to further complicate the limit states Standard by the mandatory use of new concepts in beam-column design.

In the Commentary to Clause 13.7, the linearity of interaction expressions presented in that clause was discussed. In fact the interaction curve relating axial load and the two orthogonal moments is not linear, for either strength or stability.

The simplest demonstration of non-linearity is that used in Reference 99 on work on HSS. Consider a circular tube subject to eccentric loading about the x and y axes. Because the section properties are uniform about the polar axis, the interaction expressions of Clause 13.7 would sum the moment effects as if a moment of $P(e_x + e_y)$ were applied, whereas the actual moment is

$$P\left(e_x^{\,2} + e_y^{\,2}\right)^{1/2}$$

Thus, the interaction curve for a circular tube, is a quadrant of a circle. Finding a similar effect applied to square HSS, it was determined that the available test data fitted the expression in Clause L3(c) if the sum of the moment terms was modified by the expression

$$\gamma = \frac{e_x^{\,2} + e_y^{\,2}}{e_x + e_y}$$

The value of γ is equivalent to ν in Clause L2(c).

Research at Lehigh[100,101,102,103] shows that a similar non-linear effect applies to W-shapes. The expressions in Clause L1(a) and (b) are for strength and stability respectively. The exponents ζ and η were evaluated from rigorously calculated interaction curves.

The expressions for M_{ox} and M_{oy} have been determined from Clause 13.7.1, transposed and applied for uniaxial bending only.

Essentially, the interaction expressions (a) and (b) in Clause L1 define an interaction surface relating the boundary conditions on the three perpendicular axes (i.e. axial resistance and moment resistance about each axis). Therefore, the better these values are determined, the better will be the interaction surface.

Unlike W-shapes, significant biaxial bending is imposed on HSS whenever beams frame into the column about two perpendicular axes. The design expressions given in Clause L2 for square HSS sections, for both strength and stabililty are developed in References 104, 105 and 116. Since the form of the expressions is completely different to those for W-shapes research is underway to develop exponents for the curvilinear interaction expressions for both square and rectangular HSS.

Limitations

Significant economy may result in designs according to Appendix 'L', but before adopting these, some general thought should be given to the extent to which yielding under service load is likely and, in view of this, whether further restrictions on the design are necessary. (See the Note to Appendix L.)

The Standard recommends a value of 0.85 for ω for compression members in frames subject to joint translation (sidesway). This approach should not be used in combination with the method recommended here, the development of which is based on constant end eccentricities up to maximum load. To use the recommended method in sway frames, the end moments should be determined by a second-order analysis, i.e. the $P\Delta$ effects at ultimate load should be included. Additional precautions have been proposed in Reference 106.

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FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

Feet^2 (ft^2) x 0.0929030	= Meters^2 (m^2)
Acres x 4046.86	= Meters^2 (m^2)
Miles^2 (mi^2) x 2.58999	= Kilometers^2 (km^2)

VOLUME

Feet^3 (ft^3) x 0.02831685	= Meters^3 (m^3)
Feet^3 (ft^3) x 28.31685	= Liters**
Yards^3 (yd^3) x 0.764555	= Meters^3 (m^3)
Acre feet x 1233.48	= Meters^3 (m^3)
Gallons (gal) x 0.00454609	= Meters^3 (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

Feet^3 per second (ft^3/s) x 0.02831685	= Meters^3 per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= Meters^3 per second (m^3/s)
Acre-feet per day x 0.0142764	= Meters^3 per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= Meters^3 per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 (lb-s/ft^2) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

---

* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

NOTES

Caribbean
Uniform
Building
Code
(CUBiC)

Part 2
SECTION 8

Structural Design Requirements
STRUCTURAL TIMBER

1985

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 8
STRUCTURAL TIMBER

Caribbean Community Secretariat
Georgetown
Guyana

PART 2

SECTION 8

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FOREWORD

The species and quality of timber used in the Commonwealth Caribbean territories vary widely.

Some territories (e.g. Guyana) have large stocks of locally grown timber for structural purposes. Other countries import virtually all their structural timber, while still others have a growing local timber industry which contributes a portion of the total amount of timber used annually.

Timber strengths vary even within species, depending on the conditions and locality of growth.

Recommended permissible stresses must therefore be based on standard grading rules developed from correlation between visual inspection and strength tests. Permissible stresses put forward in this document are based on North American grading rules, ASTM in particular. Stresses for structural softwood are given for Southern Yellow Pine which is fairly typical for softwoods of the pitch pine variety.

It is suggested in the document that reference be made to the local Bureau of Standards or Forest Products Laboratory in each territory for confirmation and guidance on basic stresses for timber produced in that territory.

Most of the published and unpublished data available on strengths of Caribbean timbers requires further verification, statistical analysis and collation to conform with the format presented in sub-sections 2.808 through 2.822.

While awaiting such further work, it was considered useful to publish such data as shown in Appendix D for approximate guidance and comparison. Only densities have been included at this stage.

The document is based on two design approaches:

(1) A rational design procedure using working stress design methods.

This is based on National Design Specification for Wood Construction by the National Forest Produces Association, USA.

(2) A simplified design procedure based on the use of load tables and the selection of minimum recommended sizes for various applications.

This is based on the work by the Guyana Association of Professional Engineers as a contribution to earlier efforts at a Caribbean Uniform Building Code.

For a rational design procedure for special cases such as trusses, built-up sections, and glued laminated members, reference is made direct ot the National Design Specification.

Safe loads and design values have been omitted for the wide range of connector systems available. These values are generally empirical based on actual tests on various species of timber. Inclusion of such data will be considered later after review, when the likely range of timbers will be more clearly defined.

Acknowledgement is hereby made of the many standards and other references which provided material for the preparation of this Section. Appendix B provides a partial list.

NUMBERING SYSTEM

For this part of the Code, the numbering system is as follows:

The first number indicates the Part of the Code, the first digit in the second number indicates the Section in the Part, the second and third digits in the second number indicate the sub-section in the Section, and the third number indicates the Article in the sub-section. These are illustrated as follows:

2	Part 2
2.500	Part 2, Section 5
2.506	Part 2, Section 5, sub-section 6
2.506.3	Part 2, Section 5, sub-section 6, Article 3

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 8
STRUCTURAL TIMBER

PART 2
SECTION 8

2.800 STRUCTURAL TIMBER - GENERAL

2.801 Scope

2.801.1 PRACTICE DEFINED - This specification defines the practice to be followed in structural design with

- visually graded lumber
- structural glued laminated timber

2.801.2 Rational stress design procedures are set out in Section 8A sub-sections 2.808 through 2.822.

2.801.3 Simple design procedures for flexural members using load tables are set out in Section 8B.

2.801.4 In this Specifcation the word "shall" is mandatory and the word "should" is advisory.

2.801.5 This Specification is based on the best data and engineering judgement available. However, it is not intended to preclude use of materials, assemblies, structures, or designs, not meeting the criteria herein, where it can be demonstrated by analysis based on generally recognized theory, full-scale or prototype loading tests, studies of model analogues, or extensive experience in use, that the material, assembly, structure or design can perform satisfactorily in its intended end use.

2.801.6 COMPETENT SUPERVISION - The design values for wood products and fastenings and requirements for structural design of wood construction given in this Specification are for designs made and carried out under competent supervision.

2.802 Definitions

The following words and terms shall for the purpose of this Code, have the meanings set forth in this Section.

Glued Built-up Members - Structural members, the sections of which are composed of built-up sawn lumber alone, plywood alone, or plywood in combination with sawn or glued-laminated lumber; all parts bonded together with adhesives.

Glued-laminated Lumber - Lumber composed of an assembly of wood laminations bonded with adhesives, in which the lamninations are too thick to be classed as veneers. (See definition of Structural Glued-laminated Lumber).

Grade - The classification of lumber in regard to strength and utility in accordance with the standards in the appendices.

Grade (Stress) - A lumber grade defined in such terms that a definite working stress may be assigned to it as set forth in the standards in the appendices.

Joist - One of a series of horizontal members, with narrow face up, to which floor boards or roof sheathing are fastened.

Nominal Size - The commercial size designation of width, and depth, in standard sawn lumber and glued-laminated grades - somewhat larger than the standard net size of dressed lumber, in accordance with the standards in the appendices.

Plywood Construction - A built-up panel of laminated veneers conforming to the standards given in the appendices.

Scantling - A term applied to a piece of lumber 100 to 200 millimetres thick by 100 to 210 millimetres wide.

Siding - The external wall material covering the frame of a building. This is also referred to as cladding.

Sill - A horizontal length of timber on which the studs rest, fixed to the floor. The width of the sill is the same as that of the studs.

Spar - A common roof rafter.

Structural Glued-laminated Lumber - Any member comprising an assembly of laminations of lumber in which the grain of all laminations is approximately parallel longitudinally and in which the laminations are bonded with adhesives; and which is fabricated in accordance with the standards given in the appendices.

Stud - One of a series of slender wood structural members placed vertically as supporting members in a wall, partition, or similar structural unit.

2.803 Symbols and Notations

Except where otherwise noted, the symbols used in this Specification have the following meanings:

A = area of cross section.

A_1 = in fastener group analysis, cross-sectional area of main wood member(s) before boring or grooving.

A_2 = in faster group analysis, sum of cross-sectional areas of wood or metal side member(s) before boring or drilling.

b = breadth (width) of rectangular member.

C_c = curvature factor.

C_F = size factor.

C_f = form factor.

C_k = for bending members, largest value of slenderness factor C_s at which intermediate beam formula applies when determining design value for extreme fiber in bending, defined as

$$C_k = .811\sqrt{E/F_b}$$

C_s = slenderness factor for bending member.

C_T = buckling stiffness factor.

C_x = spaced column fixity factor.

c = distance from neutral axis to extreme fiber.

D = diameter.

d = depth of rectangular member, or least dimension of rectangular compression member, inches, or pennyweight of nail or spike.

d' = depth of member remaining at a notch.

d_e = effective depth of member at a joint detail, as defined in clause 2.813.6(e).

d_1 = cross-sectional dimension of rectangular compression member in first plane of lateral support, or least dimension of cross-section of individual member of a spaced column.

d_2 = cross-sectional dimension of rectangular compression member in second plane of lateral support, or cross-sectional dimension of wide face of individual member of a spaced column.

E = Modulus of elasticity.

e = eccentricity.

F_b = design value for extreme fiber in bending.

F_b' = design value for extreme fiber in bending, adjusted by slenderness factor.

fb = actual unit stress at extreme fiber in bending.

F_c = design value in compression parallel to grain.

F_c' = design value in compression parallel to grain, adjusted for l_e/d ratio.

f_c = actual unit stress in compression parallel to grain induced by axial load.

F_c = design value in compression perpendicular to grain.

f_c = actual unit stress in compression perpendicular to grain.

F_g = design value for end grain in bearing parallel to grain.

f_g = actual unit stress in end grain in bearing parallel to grain.

F_n = design value in compression at an angle to grain.

F_r = design value in radial stress.

f_r = actual radial stress across the grain.

F_{rc} = design value in radial compression across the grain.

f_{rc} = actual unit stress in radial compression across the grain.

F_{rt} = design value in radial tension across the grain.

f_{rt} = actual unit stress in radial tension across the grain.

F_t = design value in tension parallel to grain.

f_t = actual unit stress in tension parallel to grain.

F_v = design value in horizontal shear.

f_v = actual unit stress in horizontal shear.

G = specific gravity.

g = gauge of screw.

I = moment of inertia.

J = unitless convenience factor defined as

$$J = \frac{(l_e/d) - 11}{K \quad - 11}$$

K = for columns the smallest slenderness ratio l_e/d at which the long column formula applies for determining design value in compression parallel to grain and, for fastenings, modification factor for design value for row of connector units, bolts or lag screws loaded in shear.

K_e = effective buckling length factor for columns.

K_r = radial stress factor.

L = span length of bending member, or distance between points of lateral support of column.

l = span length of bending member, or distance between points of lateral support of column, or length of bolt in main member.

l_e/d = slenderness ratio of compression member.

l_b = length of bearing in compression perpendicular to grain.

l_e = effective span length of bending member, or effective length of compression member.

l_u = laterally unsupported span length of bending member.

l_1, l_2 = distances between points of lateral support of compression member in planes 1 and 2.

l_3 = in spaced column, distance from center of connectors in an end block to center of spacer block.

M = bending or resisting moment.

N = load on a fastener or fastener group at an angle to grain between 0^o and 90^o, from Hankinson Formula.

P = total concentrated load, or total axial load, or load parallel to grain on a fastener or fastener group.

P_r = design value for a row of fastenings.

P_s = summation of design values for individual fasteners in a row.

P/A = axial load per unit of cross-sectional area.

psi = pounds per square inch.

Q = statical moment of an area about the neutral axis, or load perpendicular to grain on a fastener or fastener group.

R = radius of curvature.

R_m = for a curved glued laminated bending member having a varying rectangle cross-section, radius of curvature at mid-depth at apex.

r = least radius of gyration of section.

S = section modulus, or spacing of beams.

t = thickness.

V = shear force.

W = total uniform load.

w = uniform load per unit of length or surface area.

x = distance from beam support to load.

β = an angle of slope of upper face of curved glued laminated bending member having a varying rectangular cross-section, degrees.

θ = angle between direction of load and direction of grain, degrees.

π = pi

^oF = temperature in degrees Fahrenheit.

// = parallel.

$\perp$ = perpendicular

$>$ = greater than

$\geqslant$ = greater than or equal to

$<$ = less than

$\leqslant$ = less than or equal to

2.804 Species

2.804.1 This Code is based on timbers likely to be available in the Caribbean. However, since both local and imported timbers are likely to vary so considerably throughout the region it is not possible to include an exhaustive list. Table D1, Appendix D is a partial list.

2.804.2 For the use of timbers, not listed, reference should be made to local Forest Products Research Laboratories or other recognised testing authorities.

2.804.3 Some further information on Caribbean timber is included in Table 2.804.1. The information in Appendix D is not definitive and is included as interim information for guidance only.

TABLE 2.804.1
NAMES AND DENSITIES OF SOME STRUCTURAL TIMBERS AVAILABLE
(LOCAL OR IMPORTED IN THE COMMONWEALTH CARIBBEAN*)

Standard Name	Botannical Species	Other Common Names	Approximate Density at a Moisture Content of 18%
Softwoods			kgf/m^3
Douglas Fir	Pseudotsuya taxifolia	B.C. Pine	590
Pitch Pine	Pinus palustris Pinus ellioti Pinus Caribea	Long leaf pitch pine, southern yellow pine, Nicaraguan pitch pine, Honduras pitch pine	720
Redwood	Pinus sylvestris	Redwood	544

TABLE 2.804.1 (continued)

Whitewood	Picea abies	Baltic whitewood	512
	Abies alba	European whitewood	448
Hardwoods			
African Mahogany	Khaya spp.	Khaya	590
Greenheart	Ocotea rodiaei		1,030
Jarrah	Eucalyptus marginata		912
Sapele	Entandrophragma cylindricum		688

NOTE: An extended list of unverified data on Caribbean timbers is included in Table D1 Appendix D for preliminary comparisons only. Users of the Code should refer to their local forest Products Laboratory or Bureau of Standards for accurate local data.

2.805 Grading

2.805.1 LUMBER - The design values for visually graded lumber are based on the provisions of ASTM Designation D245, "Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber." These methods involve adjusting the strength properties of small clear specimens of wood as given in ASTM Designation D2555, "Methods for Establishing Clear Wood Strength Values", for the effects of knots, slope of grain, splits, checks, size, duration of load, moisture content and other influencing factors to obtain design values applicable to normal conditions of service.

2.805.2 The appropriateness of lumber design values is regularly evaluated on the basis of experience and experimental data developed on the properties and performance of clear wood specimens or full size lumber pieces.

2.805.3 In the Caribbean there is a wide range of species of lumber, particularly hardwoods, many of which would not be included in the experimental programmes of the ASTM. For such cases reference should be made to local Forestry Departments.

2.805.4 Other authoritative reference sources are "The Guyana Grading rules for Hardwood Timber: published by the Forestry Department, Georgetown, Guyana and "Strength Properties of Trinidadian Timber" published by the University of the West Indies.

2.805.5 Design values for machine stress reated (MSR) lumber are based on non-destructive stiffness testing of individual pieces. Certain visual grade requirements also apply to such lumber. The stress-rating system used for MSR lumber should be regularly checked by the responsible grading agency for conformance to established certification and quality control procedures. An example of such a responsible agency in the region is the applicable Bureau of Standards.

2.805.6 GLUED LAMINATED TIMBER - Design values for Glued Laminated Timber shall be accordance with those published by the American Institute of Timber Construction (AITC).

2.806 Moisture Content of Timber

2.806.1 The timber should be seasoned as far as practicable to a moisture content appropriate to the position in which it is to be used. "Dry" basic stresses given in this document assume a moisture content not exceeding 19% when full design loads are applied. This is considered to be a normal service condition for most covered structures.

2.806.2 For lumber of 50 mm and 100 mm minimum dimensions 15% moisture content is considered a more normal dry service condition and where such lumber is manufactured at 15% moisture content higher stresses are generally allowed.

2.806.3 For glued laminated timber, the corresponding dry service moisture contents are reduced further to a maximum of less than 16%.

2.806.4 In some countries of the Caribbean varying conditions of humidity etc. will cause some variation in normal service. Where local vlaues vary more than 5% from the values stated here, the recommendations of the local Forest Products Laboratory or Bureau of Standards shall be sought.

2.806.5 A rough adjustment would be to reduce the wet service condition (19% moisture content) allowable stresses by 2 1/2% for each 1% increase in moisture content.

2.807 Sizes

2.807.1 Wood members shall be of sufficient size to carry the dead, live, wind and earthquake loads, without exceeding the allowable deflections or working stresses specified in this Code.

2.807.2 Sizes of wood members referred to by the Code are nominal. Nominal sizes may be shown on the plans. the minimum acceptable net sizes conforming to nominal sizes shall be within 2% of the minimum net sizes contained in Appendix A at 19% moisture content.

2.807.3 Minimum sizes shall be checked on site.

2.807.4 Appendix A shows standard nominal and corresponding dressed sizes for structural lumber.

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 8A
STRESS DESIGN PROCEDURES

SECTION 8A

2.808 STRESS DESIGN PROCEDURES FOR SAWN TIMBER

2.809 Alternative to use of Load Tables

2.809.1 Wood joists and rafters may be of the sizes set forth in the load table, Table 2.825.1 Section 8B without additional structural design.

2.809.2 Alternatively they may be designed using the allowable stresses set out in this Section 8A.

2.810 Effective Cross-section

2.810.1 The net section, obtained by deducting from the dressed section, the projected area of all material removed by boring, grooving, dapping, notching or other means shall be used in calculating the load carrying capacity of a member.

2.810.2 The effects of any eccentricity of loads applied to the member at that section should be taken into account.

2.810.3 Computations to determine the required sizes of members shall be based on net sizes contained in Appendix A.

2.811 Basic Stresses

2.811.1 Basic stresses or design values for a selection of species are given in the following Table No. 2.811.1

2.811.2 These are based on National Design Specification by US National Forest Products Association. The list is by no means exhaustive and is intended to include those species known to be used in fair quantities in the region and those for which such stress information is available. The list is extended to Appendix D for Caribbean lumbers for which only limited information is currently available.

2.811.3 Local Forest Products Association should be consulted to obtain information on other species.

2.811.4 The values in Table 2.811.1 relate to end grain bearing values for end grain bearing parallel to grain on a rigid surface.

2.811.5 For stress consideration, such as extreme fiber in bending, tension parallel to grain, horizontal shear, compression perpendicular to grain and design values will depend on the grade of timber within a particular species. These values are shown in Table 2.812.1.

TABLE 2.811.1
BASIC DESIGN VALUES (END GRAIN IN BEARING) N/mm^2

		Dry service conditions[1]		
		Sawn lumber[2]		
Species	Wet service conditions[1]	More than 100mm thick	Not More than 100mm thick	Glued laminated timber
California Redwood Open grain	8	8.7	11.5	13.3
Douglas Fir South	8.5	9.2	12.5	14.6
Southern Pine	9	10	13.5	15.8

1. Wet and dry service conditions are defined in sub-sections 2.811 and 2.812 for sawn lumber and Ref. No. 1 for glued laminated timber.

2. Applies to sawn lumber members which are at a moisture content of 19 percent or less when full design load is applied, regardless of moisture content at time of manufacture.

3. When 100 mm or thinner sawn lumber is surfaced at a moistrue content of 15 percent or less and is used under dry service conditions, the values listed for glued laminated timber may be applied.

4. Appendix D gives approximate values for some Caribbean Timbers for comparison.

2.812 Permissible Stress and Modification Factors

2.812.1 PERMISSIBLE STRESSES - Permissible stresses in timber of any particular species are governed by the particular conditions of moisture, temperature, preservative and fire retardant treatment and duration.

2.812.2 Table 2.812.1 shows recommended permissible stresses for the group of softwoods which include Douglas Fir, Pitch-pine (including Southern Yellow Pine).

2.812.3 For other types of softwood reference can be made to U.S. National Design Specifications for recommended permissible stress. Appendix D shows some limited information on some Caribbean timbers.

2.812.4 Modifications are required for the various conditions described above and these are set out in clause 2.812.5.

TABLE 2.812.1
PERMISSIBLE STRESSES FOR VISUALLY GRADED LUMBER UNDER NORMAL CONDITIONS OF LOADING
N/mm^2

Species and Commercial Grade	Southern Pine No. 2	Southern Pine No. 2
Size Classification	50 mm to 100 mm wide	50 mm to 100 mm thick 127 mm and wider
Extreme fiber in bending, F_b		
Single member uses	10.7	9.0
Repetitive member uses	12.0	10.3
Tension parallel to grain, F_t	6.20	4.65
Horizontal Shear, F_v	0.65	0.65
Compression perpendicular to grain, F_c	3.90	3.90
Compression parallel to grain, F_c	7.9	8.30
Modulus of Elasticity, E	11,000	11,000

Grading rules agency: Southern Pine Bureau

It is assumed that this lumber is surfaced at 15% maximum moisture content, K.D.-15. Used at 15% M.C.

Appendix D gives some values in varying formats available for Caribbean Timber.

2.812.5 MODIFICATION FACTORS

(a) Moisture Service Conditions:

The modification of tablulated or defined design values to account for different wood strength at different moisture service conditions is specified for sawn lumber, glued laminated timber, along with the tables of permissible stresses in this section or from Reference No. 1.

(b) Temperature:

The permissible stress presented herein are applicable to members used under ordinary ranges of temperature and occasionally heated in use to temperatures up to 150^{o} F. Wood increases in strength when cooled below normal temperatures and decreases in strength when heated. Members heated in use to temperatures up to 150^{o} F will return essentially to original strength when cooled. Prolonged temperatures above 150^{o} may result in permanent loss of strength. Some reduction in design values may be necessary in specific applications to account for the temporary decrease in strength occurring when members are heated to elevated temperatures up to 150^{o} F for extended periods of time. Information on the approximate immediate effect of temperature on mechanical properties is given in Reference No. 1.

(c) Preservative Treatment:

The permissible stress values provided herein apply to wood products pressure-impregnated by an approved process and preservative.

(d) Fire-retardent-treated Wood:

(i) For lumber pressure-impregnated with fire-retardant chemicals, the design values otherwise permitted herein shall be reduced 10 percent.

(ii) The design values for structural glued laminated timber pressure-impregnated with fire-retardant chemicals before or after gluing are dependent upon the species and treatment combinations involved. The effect on strength must be determined for each treatment. The manufacturer of the treatment should be contacted for specific information on fire-retardant adjustments for all recommended design values. The resulting values are subject to duration of load adjustments as set forth in Reference No. 1.

(e) Duration of Load:

(i) Wood has the property of carrying substantially greater maximum loads for short durations than for long durations of loading. Recommended design values provided herein apply to normal duration of loading. Normal load duration contemplates a load that fully stresses a member to its allowable design value by the application of the full design load for a duration of approximately ten (10) years, either continuously or cumulatively. For other than normal duration of loading, design values for wood members, and design values for fastenings when fastening load capacity is determined by the strength of the wood rather than the strength of the metal, shall be adjustd in accordance with sub-clause e(ii) to e(vii) to take into account the change in strength of wood with changes in duration of loading.

(ii) When a wood member will be fully stressed to the maximum design stress or a joint will be fully loaded to the maximum design load for a total of more than ten years, either continuously or for cumulative periods of maximum design stress or load, 90 pecent of the normal design values otherwise permitted herein shall apply. (See sub-clause e(vi) and e(vii)).

(iii) When the duration of the full maximum load does not exceed the period indicated, the normal design values for wood members and fastenings shall be multiplied by the following modification factors (see sub-clauses e(iv), e(vi) and e(xii).

1.25 for 7 days duration of load;
1.33 for wind or earthquake;
2.00 for impact.

(iv) For members pressure-impregnated with preservative salts to the heavy retentions required for "marine" exposure, and for lumber pressure-impregnated with fire-retardant chemicals, the impact load duration factor in sub-clause e(iii) shall not apply. For impact normal design values shall be used.

(v) The modification factors in sub-clauses e(ii) and e(iii) are not cumulative. The resulting sizes of structural members or load carrying capacities of joints shall not be smaller than required for a lesser design load acting for a longer duration.

(vi) The provisions of sub-clauses e(i) to e(v) apply to mechanical fastenings when fastening load capacity is determined by the strength of the wood rather than the strength of the metal, unless otherwise provided.

(vii) The provisions of sub-clauses e(i) to e(vi) do not apply to modulus of elasticity.

(viii) The provisions of sub-clauses e(i) to e(v) do not apply to "compression perpendicular to grain" design values based on a deformation limit.

(f) Flexure:

Modification of permissible stresses for slenderness, size factors, orientation is set out in sub-section 2.813 on member design.

2.812.6 BEARING ON END GRAIN - Basic stresses set out in Table 2.811.1 may be taken as permissible stresses for bearing on end grain.

2.812.7 When end grain bearing stresses exceed 75% of these basic stresses, the bearing shall be on a metal plate or on other durable, rigid, homogeneous material of adequate strength. For end to end bearing, adequate lateral support must be provided and where required the rigid inserts shall be of not less than 20 gauge metal plate or equivalent, with a snug fit between abutting ends.

2.812.8 BEARING AT AN ANGLE TO GRAIN - When the load at bearing is at an angle to grain, the maximum bearing value shall be determined by Hankinson's Formula in accordance with sub-section 2.818, Stresses and Loads at an Angle to Grain.

2.813 Member Design

2.813.1 When the design values specified herein are used, the lumber shall be identified by the grade mark of, or certificate of inspection issued by a lumber grain or inspection bureau or agency, recognised as being competent.

2.813.2 The design values are specified for lumber that will be used under dry conditions of service such as in most covered structures.

2.813.3 Provision is made for two conditions of dry service as follows:

(a) Where the moisture content in use will be a maximum of 19 percent, regardless of the moisture content at the time of manufacture, and,

(b) Where the moisture content in use will be a maximum of 15 percent, applicable to 50 mm - 100 mm thick lumber that was manufactured at a maximum moisture content of 15 percent.

2.813.4 Lumber sizes referred to are nominal sizes. Computations shall be based on minimum dressed sizes (Appendix A). For the dressed sizes, dimensions set out in Appendix A may be used. Minimum sizes shall also be verified during construction.

2.813.5 FLEXURAL MEMBERS - BENDING

(a) Beam Span:

For simple beams, the span shall be taken as the distance from face to face of supports, plus one-half the required length of bearing at each end; for continuous beams, the span shall be measured to the centres of bearings on supports over which the beam is continuous.

(b) Notches:

Notches in sawn lumber bending members shall not exceed one-sixth the depth of the member and shall not be located in the middle third of the span. Where members are notched at the ends, the notch depth shall not exceed one-fourth the beam depth. The tension side of sawn lumber bending members of 100 mm or greater nominal depth shall not be notched, except at ends of members.

(c) Flexural Design Formulae:

The stress at extreme fibre in bending, induced by bending moment M is calculated by the formula:

$$f_b = \frac{M}{S}$$

where S = Section Modulus

For a rectangular bending member of width b and depth d.

$$S = \frac{bd^2}{6}$$

and $f_b = \frac{M}{S} = \frac{6M}{bd^2}$

(d) Lateral Support of Bending Members:

Where full allowable bending stress F_b is to be used, rectangular sawn lumber beams, rafters and joists shall be laterally supported as follows:

For rectangular lumber beams, rafters, joists or other bending memebers, the designer should apply the following approximate rules in providing restraint to prevent rotation or lateral displacement. If the ratio of depth to thickness, based on nominal dimensions, is:

(i) 2 to 1; not lateral support is required.

(ii) 3 to 1 to 4 to 1; the ends shall be held in position, as by full depth solid blockng, bridging, hangers, nailing or bolting to to other framing members, or by other acceptable means.

(iii) 5 to 1; the compression edge shall be held in line for its entire length and ends held in position.

(iv) 6 to 1; bridging, full depth solid blocking or cross bracing shall be installed at intervals not exceeding 2.5 m unless both edges are held in line or unless the compression edge of the member is supported throughout its length to prevent lateral displacement, as by adequate sheathing or sub-flooring, and the ends at points of bearing have lateral support to prevent rotation.

(v) 7 to 1; both edges shall be held in line for their entire length.

If a bending member is subject to both flexure and compression parallel to grain, the depth to breadth ratio may be as much as 5 to 1 if one edge is held firmly in line. If under all combinations of load, the unbraced edge of the member is in tension, the ratio may be 6 to 1.

(e) Slenderness Factor and Flexural Stress:

(i) When the depth of a binding member does not exceed its breadth, no lateral support is required and the design value otherwise permitted at extreme fibre in bending may be used.

(ii) When the depth of a bending member exceeds its breadth, lateral support shall be provided at points of bearing to prevent rotation and/or lateral displacement at those points, and the design value at extreme fibre in bending shall be modified as required by use of the slenderness factor C_s.

(iii) When the compression edge of a bending member is supported throughout its length to prevent its lateral displacement, and the ends at points of bearing have lateral support to prevent rotation, the unsupported length, l_u, may be taken as zero.

When lateral support is provided to prevent rotation at the points of end bearing, but no other lateral support is provided throughout the length of the bending member, the unsupported length, l_u, is the distance between such points of end bearing, or the length of a cantilever.

When a bending member is provided with lateral support to prevent rotational and/or lateral displacement at intermediate points as well as at the ends, the unsupported length, l_u, is the distance between such points of intermediate lateral support.

(iv) The slenderness factor for a bending member shall be calculated by the following formula:

$$C_s = \frac{l_e d}{b^2}$$

where l_e = effective length
b = breadth

For single span beam with concentrated load at center,

$$l_e = 1.61\ l_u$$

For single span beam with uniformly distributed load,

$$l_e = 1.92\ l_u$$

For single span beam with equal end moments,

$$l_e = 1.84\ l_u$$

For cantilever beam with concentrated load at unsupported end,

$l_e = 1.69\ l_u$

For cantilever beam with uniformly distributed load,

$l_e = 1.06\ l_u$

For single span or cantilever beam with any load, conservative value for $l_e = 1.92\ l_u$

The effective lengths, l_e, above are based on an l_u/d ratio of 17. For other l_u/d ratios, these effective lengths may be multiplied by a factor equal to $0.85 + 2.55/\ (l_u/d)$ except that this factor shall not apply to a single span beam with equal end moments ($l_e = 1.84\ l_u$) or to a single span or cantilever beam with any load ($l_e = 1.92 l_u$).

(v) The slenderness factor, C_s, shall not exceed 50.

(vi) The design values for extreme fibre in bending, F_b, and modulus of elasticity, E, used in the formulas in sub-clause e(viii), e(ix) and (x) shall be modified in accordance with the provisions in sub-section 2.812 except that the modification for size factor shall not apply to F_b in (ix) and (x).

(vii) Design values for extreme fibre in bending adjusted for slenderness factor F_b', obtained from (viii) to (x) are not subject to further modifications for moisture service condition, duration of loading, temperature, type of treatment or size.

(viii) Short Beams. When the slenderness factor, C_s, does not exceed 10, the full design value for extreme fibre in bending, $F_b' = F_b$ may be used.

(ix) Intermediate Beams - When the slenderness factor, C_s, is greater than 10, but does not exceed C_k, the design value for extreme fibre in bending, F_b, shall be determined from the following formula:

$$F_b' = F_b\left[1 - \frac{1}{8}\left(\frac{C_s}{C_k}\right)^4\right]$$

in which $C_k = .811\sqrt{E/F_b}$

(x) Long Beams - When the slenderness factor, C_s, is greater than C_k, but less than or equal to 50, the design value for extreme fibre in bending, F_b shall be determined by the following formula:

$$F_b{}' = \frac{0.438E}{(C_s)^2}$$

(xi) The design value for extreme fibre in bending, F_b determined in accordance with (ix) and (x) shall not exceed the full design value for extreme fibre in bending, F_b modified in accordance with sub-section 2.812 (including the modification for size factor).

(f) Form Factor:

(i) Formula - for bending members with a circular cross-section, or square cross-section loaded in the plane of the diagonal, the resisting moment shall be modified by a form factor C_f, as in the following formula:

$$M = \frac{C_f F_b I}{c}$$

(ii) Rounded Beams - the form factor, C_f, of a bending member with a circular cross-section is 1.18; such a member thus has the same moment capacity as a square bending member having the same cross-sectional area. If a circular member is tapered, it shall be considered a beam of variable cross-section.

(iii) Diamond Section Beam - the form factor, C_f, of a square bending member that is loaded in the plane of a diagonal is 1.414; such a member thus has the same moment capacity whether oriented with a side or a diagonal parallel to the load direction.

The form factor shall be cumulative with the size factor.

2.813.6 FLEXURAL - SHEAR

(a) Design Formulas for Shear Strength Along-the-Grain:

The along-the-grain (horizontal) shear stress induced in a sawn lumber, glued laminated timber or timber bending member shall be calculated by means of the formula:

$$f_v = \frac{VQ}{Ib}$$

For a rectangular bending member of width b and depth d, this becomes:

$$f_v = \frac{3V}{2bd}$$

(b) Design Stresses:

Except as provided in sub-clauses (c), (e) and (d) the unit horizontal shear stress f_v at any cross-section of the bending member shall not exceed the design value for the product, species and grade, in Table 2.811.1 and modified in accordance with sub-section 2.812 and such other sections as may apply.

(c) Horizontal Shear in Notched Beams:

For notched bending members, the shearing strength of a short relatively deep member notched on the lower face at the end is decreased by an amount depending on the relation of the depth of the notch to the depth of the member. For limits to notch depths see clause 2.813.5.

the desired bending load shall be checked against the load obtained by the formula:

$$V = \frac{(2F_v bd')}{3} \quad \frac{(d')}{d}$$

d' is depth of member remaining at the notch.

A gradual change in cross-section compared with a square notch increases the shearing strength nearly to that computed for the actual depth above the notch.

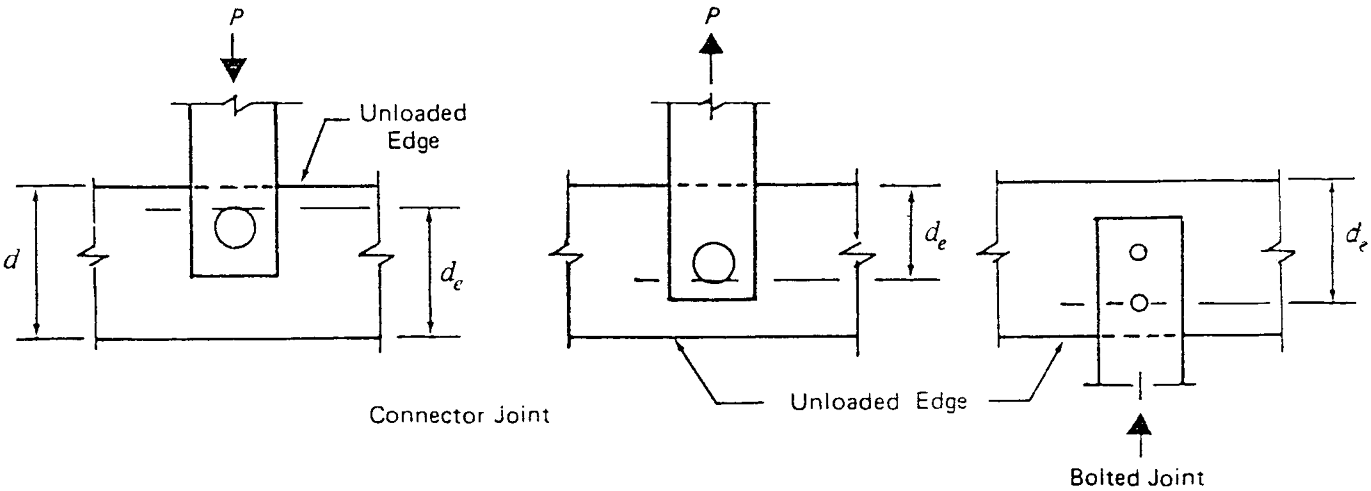
(d) Design Values for Shear in Joints:

Design values for shear in joint details, as shown in 2.813.1 may be 50 percent greater than the design values in shear otherwise permitted, provided the joint is at least 5 times the depth of the member from its end.

(e) Design of Joints in Shear:

(i) Joints in shear, fastened with connectors (split rings, shear plates), bolts or lag screws (including beams supported by such fasteners of other cases as shown in Fig. 2.183.1) shall be designed so that f_v, in the following formulae does not exceed the design value in horizontal shear.

FIGURE 2.813.1
SHOWING d_e FOR MEMBER WITH VARIOUS FASTENINGS



When the joint is at least five times the depth of the member from its end, the induced shear stress is calculated as:

$$f_v = \frac{3V}{2bd_e}$$

and the design values may include the 50 percent increase for shear in joint details permitted in 2.813.6 (d)

When the joint is less than five times the depth of the member from its end, the induced shear stress is calculated as:

$$f_v = \frac{3V}{2bd_e} \quad \frac{d}{d_e}$$

and the 50 percent increase in design values for shear in joint details does not apply.

The symbol d_e means:

d_e (with connectors) = the depth of the member, less the distance from the unloaded edge of the member to the nearest edge of the nearest connector.

d_e (with bolts or lag screws only) = the depth of the member, less the distance from the unloaded edge of the member to the center of the nearest bolt or lag screw.

(ii) For joint details involving mechanical fastenings as shown in Figure 2.813.1 total shear at the joint shall be limited such that the induced unit stress, f_v, determined on the basis of the full cross-section, does not exceed the design value in shear without the 50 percent stress increase.

(iii) When concealed hangers are used, the formula, in 2.813.6 (c) shall apply and the 60 percent increase in design values for shear in joint details does not apply.

2.813.7 FLEXURAL MEMBERS - DEFLECTION

(a) Deflection Calculations:

Deflections shall be calculated by standard methods of engineering mechanics, using the design values for modulus of elasticity from this Specification.

(b) Permanent Loading:

Under long-term loading, a seasoned wood bending member may acquire a permanent set about equal to half the initial deflection due to the long-term load. For unseasoned lumber, this set may be about equal to the initial deflection due to the long-term load. Where total deflection under permanent loading must be limited, extra stiffness can be provided by increasing member size to allow for this inelastic deformation. Total long-term deflection may be calculated as the immediate deflection due to the long-term or permanent component of the design load, times an appropriate factor to account for inelastic deformation, plus the deflection due to the short-term or normal component of the design load. The factor to account for inelastic deformation under long-term or permanent loading may be taken aas 1 1/2 for glued laminated timber or seasoned lumber, or 2 for unseasoned lumber.

2.813.8 COMPRESSION MEMBERS - GENERAL

(a) Terminology:

For purposes of this Specification, the term "column" refers to all types of compression members, including members forming part of trusses or other structural components.

(b) Net Section:

In the design of intermediate or long columns, the design value in compression parallel to grain adjusted for slenderness ratio, F_c', shall apply to the full column section, except that it shall apply to the net column section when the reduced section occurs in the critical part of the column length that is most subject to potential buckling. In addition, the unit stress induced in any column, based on the net column section, shall not exceed the design value in compression parallel to grain for a short column, F_c' at any point in the column length.

(c) Compression Members Bearing End to End:

For end-grain bearing of wood on wood, and on metal plates or strips, see clause 2.812.6 to 2.812.8.

(d) Eccentric Loading or Combined Stresses:

For columns subject to eccentric loading or combined flexural and axial loading, see sub-section 2.817.

(e) Modification of Column Design Values:

(i) The design values in compression parallel to grain F_c, and modulus of elasticity, E, used in the column formulas sub-section 2.816 shall incorporate the appropriate modifications applicable to the conditions under which the wood is used, as given in clause 2.812.5.

(ii) Design values in compression parallel to grain adjusted for slenderness ratio, F_c, obtained from the column formulas in sub-section 2.815 are not subject to further modifications for moisture service condition, duration of loading, temperature or types of treatment.

(iii) Design values in compression parallel to grain F_c, for use in column formulae in sub-section 2.815 shall be subject to the duration of load modifications in clause 2.812.5.

(iv) Design values for modulus of elasticity, E, for use in column formulae sub-section 2.815 are not subject to "duration of load modifications".

(f) Column Bracing:

Column bracing shall be installed where necessary to resist wind or other lateral forces.

2.814 Design of Simple Solid Columns

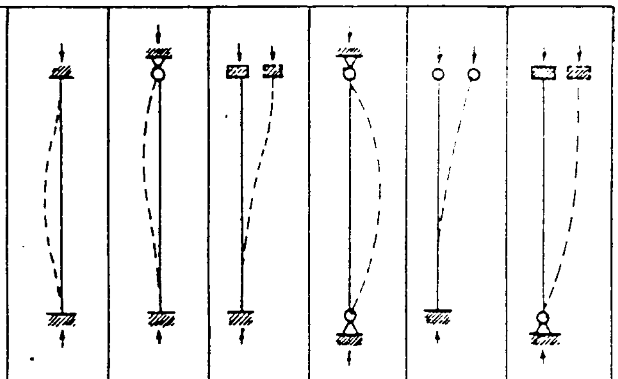
2.814.1 EFFECTIVE COLUMN LENGTH

(a) The effective column length, l_e, shall be determined in accordance with good engineering practice. Actual column lengths, l, may be multiplied by the factors given in Table 2.814.1 to determine effective column length i.e.

$$l_e = K_e l$$

(b) The effective column length, l_e, shall be used in the design formula given in this Section.

TABLE 2.814.1
EFFECTIVE COLUMN LENGTH

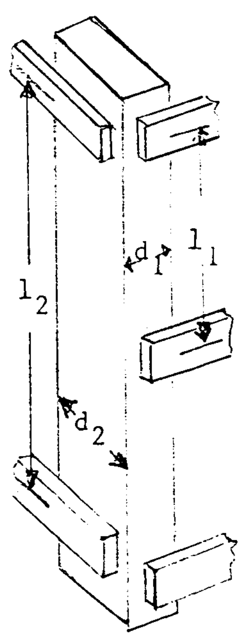
Buckling modes						
Theoretical K_e value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design K_e when ideal conditions approximated	0.65	0.80	1.2	1.0	2.10	2.4
End condition code		Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed, translation free Rotation free, translation free				

2.814.2 LIMITATIONS ON l_e/d RATIOS

(a) For simple solid columns, the slenderness ratio, l_e/d, shall be taken as the large of the ratios l_1/d_1 and l_2/d_2 where each ratio has been adjusted by the applicable factors in Table 2.814.1.

(b) For simple solid columns, l_e/d shall not exceed 50.

FIGURE 2.814.1
SIMPLE SOLID COLUMN l/d RATIOS



l_1 and l_2 =

Distance between points of lateral support of column in planes 1 and 2

d_1 and d_2

Dimensions of column in planes of lateral support

2.814.3 DESIGN VALUES FOR SIMPLE SOLID COLUMNS

(a) Maximum design values, F_c, in pounds per square inch of cross-sectional area of square or rectangular simple solid columns shall be determined in accordance with the following formulae. (See sub-sections 2.812 and 2.813 for modifications applicable to F_c, E and F_c').

Calculations done in imperial units convert to N/mm^2 by dividing by 145.

(i) Short Columns: For short columns having and l_e/d ratio of 11 or less

$$F_c' = F_c$$

(ii) Intermediate Columns: For solid columns having l_e/d ratio greater than 11 but less than K, where

$$K = 0.671 \frac{E}{F_c}$$

$$F_c' = F_c \left[1 - \frac{1}{3} \left(\frac{le/d}{K} \right)^4 \right]$$

(iii) Long Columns: For solid columns having an l_e/d ratio of K or greater

$$F_c' = \frac{0.30E}{(l_e/d)^2}$$

(iv) For especially severe service conditions and/or extraordinary hazard, use of lower design values may be necessary.

2.814.4 ROUND COLUMNS

The design load for a column of round cross-section shall not exceed that for a square column of the same cross-sectional area.

2.815 Tension Members

2.815.1 TENSION PARALLEL TO GRAIN - The unit stress in axial tension parallel to grain of wood, f_t, shall be determined on the basis of net area and shall not exceed the design values in tension parallel to grain, F_t, specified herein for sawn lumber or glued laminated timber.

2.815.2 TENSION PERPENDICULAR TO GRAIN - Because wood is relatively weaker and more variable in tension perpendicular to grain than in other properties and because this property has not been extensively evaluated, designs that induce tension perpendicular to grain stresses should be used with caution. When tension stresses perpendicular to grain cannot be avoided mechancial reinforcement sufficient to resist all such forces should be considered.

2.815.3 An example of a design that induces critical tension perpendicular to grain stresses is the practice of hanging loads below the neutral axis of a beam. This design should be avoided when heavy or medium loads are suspended from the member.

2.816 Flexural and Axial Loading Combined

2.816.1 FLEXURAL AND AXIAL TENSION - Members subjected to both flexure and axial tension shall be so proportioned that

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} \quad \text{does not exceed ONE}$$

and $$\frac{f_b - f_t}{F_b'} \quad \text{does not exceed ONE.}$$

2.816.2 For modification of F_b for size factor, see sub-section 2.812. The values of F_t and F_b shall include duration of load and other applicable modifications in sub-section 2.812.

2.816.3 FLEXURAL AND AXIAL COMPRESSION

(a) Members subjected to both flexure and axial compression shall be so proportioned that

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b'} \quad \text{does not exceed ONE}$$

Provided that l_e/d does not exceed 11.

For long columns, defined as columns with l_e/d greater than K, members must be proportioned that

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b' - f_c} = 1$$

F_c' and K are as defined in clause 2.814.3

Intermediate cases may be interpolated alternatively. (See Reference No. 1).

2.817 Design for Bearing

2.817.1 BEARING ON END GRAIN - The induced unit stres in end grain bearing parallel to grain, f_g, shall be determined on the basis of net area and shall not exceed the design values given as basic stresses in Table 2.811.1

2.817.2 BEARING PERPENDICULAR TO GRAIN

(a) The induced unit stress in bearing perpendicular to grain, F_c, shall be determined on the basis of net area and shall not exceed the compression perpendicular design values given to this Specification.

(b) The design values for compression perpendicular to grain, specified herein, apply to bearings of any length at the ends of a member, and to all bearings of 150 mm or more in length at any other location. When calculating the bearing area at the ends of members, no allowance shall be made for the fact that, as the member bends, the pressure upon the inner edge of the bearing is greater than at the end of the member.

(c) For bearings less than 150 mm in length and not nearer than 75 mm to the end of a member, the maximum load per square inch is obtained by multiplying the design values in compression perpendicular to grain by the following factor:

$$\frac{l_b + 0.375}{l_b}$$ (Calculations done in imperial units 1 mm = 0.0394 inch)

in which l_b is length of bearing in inches measured along the grain of the wood. This formula gives the following multiplying factors for indicated lengths of bearing on such small areas as plates and washers.

Length of Bearing in inches	1/2	1	1 1/2	2	3	4	6 or more
Factor	1.75	1.38	1.25	1.19	1,13	1.10	1.00

2.818 Stresses and Loads at Angle to Grain

2.818.1 COMPRESSION AT ANGLE TO GRAIN - Design values in compression on surfaces inclined to the grain, as illustrated in Figure 2.818.1 shall be determined by the Hankinson formula as provided in clause 2.818.3.

FIGURE 2.818.1

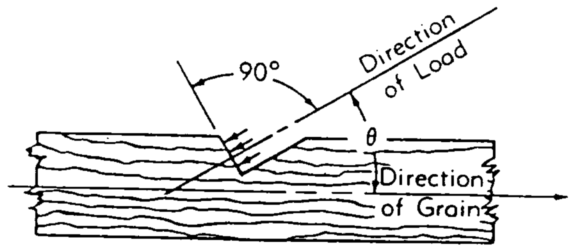
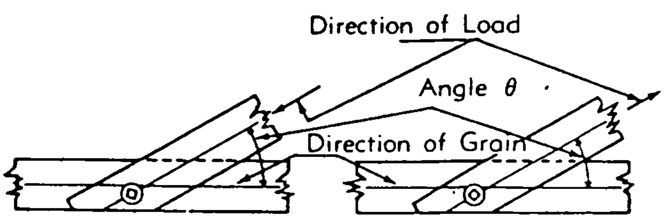


FIGURE 2.818.2



2.818.2 FASTENER LOADS AT ANGLE TO GRAIN - Design values for connectors, bolts and lag screws loaded at an angle to grain between 0^o and 90^o as illustrated in Figure 2.818.2 shall be determined by the Hankinson formula as provided in 2.818.3.

2.818.3 HANKINSON FORMULA

(a) The Hankinson Formula applies to design values in compression at an angle to grain, as follows:

$$F_n = \frac{F_c F_{c_\perp}}{F_c \sin^2\theta + F_{c_\perp} \cos^2\theta}$$

(b) The Hankinson Formula applies to design values for connectors, bolts and lag screws loaded at an angle to grain, as follows:

$$N = \frac{PQ}{P \sin^2\theta + Q \cos^2\theta}$$

2.819 Built-up Members

2.819.1 For the design of built-up members e.g. box sections, trusses, reference can be made to Reference No. 1 Appendix B.

2.819.2 Where metal is used for connecting wood members such metal should not be less than 0.9 mm thick and shall be galvanized with a zinc coating conforming to the Standards given in the Appendices.

2.820 Glued Laminated Members

2.820.1 For the design of glued laminated members reference can be made to Reference No. 1 Appendix B.

2.821 Plywood

2.821.1 Plywood lumber structural assemblies shall not exceed the working stresses set forth in the Standards in Appendix B.

2.821.2 Working stresses of plywood other than those in the Standards shall be determined according to the species.

2.821.3 Plywood sheathed shear panels may be used to resist shear due to wind and earthquake forces. For plywood 8 mm thick minimum and with continuous panel members of 50 mm minimum width and 600 mm spacing nailed with 8d nails with minimum penetration of 38 mm an allowable shear of 2.3 kN/m may be used for design.

2.822 Joints

2.822.1 Safe loads for joints in timber are largely based on empirical data derived from actual tests.

2.822.2 The Safe loads vary with bolt size, timber species and grade, member thickness, edge distances, among other considerations.

2.822.3 Users of this Code are referred to the US National Design Specifications for values of safe load for various types of fastener.

2.822.4 For nailed joints in light framed construction sub-section 2.834 sets out recommended nailing.

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 8B
PROCEDURE FOR DESIGN BY THE USE OF LOAD TABLES AND MINIMUM CONSTRUCTION DETAILS FOR FRAMED TIMBER CONSTRUCTION

PART 2
SECTION 8B

2.823 THE USE OF LOAD TABLES AND MINIMUM CONSTRUCTION DETAILS FOR FRAMED TIMBER CONSTRUCTION

2.824 Scope

2.824. Sub-sections 2.823 to 2.834 in this Section 8B are intended to provide simplified procedures for selection of members for light-framed construction. One or two storey buildings designed in accordance with this Section 8B will be deemed to satisfy all the requirements of Section 8A.

2.825 **Load Tables for Joists and Rafters of Structural Softwood**

2.825.1 Wood joists and rafters of structural softwood may be of the sizes set forth in the Table 2.825.1 without additional structural design or alternatively shall be designed based on the allowable unit stresses set forth in Section 8A.

2.825.2 The span of roof rafters shall be measured horizontally from centre to centre of bearing, and the horizontal distance from plate to ridge or other support shall be the span.

2.825.3 Where there is an accessible space above the ceiling having a clear vertical height of 750 mm or more, ceiling joists shall be designed as having usable attic space.

TABLE 2.825.1
ALLOWABLE SPANS - FLOOR JOISTS
ROOF AND CEILING JOISTS

(Based on 8.28 N/mm^2 (1200 psi) fibre stress and L/360 deflections)

Size of Joists or Rafters mm (ins)	Spacing mm (ins)	Maximum Allowable Span for Uniform Loading metres (feet - inches) 3.2kN/m^2 (67 psf)	2.7kN/m^2 (57 psf)	2.2kN/m^2 (47 psf)	1.8kN/m^2 (37 psf)	1.1kN/m^2 (22 psf)
	300 (12)	–	–	–	2.3(7-6)	2.7(9-0)
50 x 100	400 (16)	–	–	–	2.0(6-9)	2.4(8-0)
(2x4)	600 (24)	–	–	–	2.0(5-6)	2.0(6-6)
	300 (12)	3.0(10-2)	3.3(11-0)	3.4(11-3)	3.7(12-4)	5.3(17-7)
50 x 150	400 (16)	2.7(8-10)	2.9(9-6)	3.1(10-2)	3.4(11-3)	4.6(15-2)
(2x6)	600 (24)	2.2(7-2)	2.4(7-9)	2.6(8-8)	2.9(9-8)	3.8(12-5)
	300 (12)	4.1(13-6)	4.4(14-7)	4.5(15-0)	5.0(16-6)	7.1(23-6)
50 x 200	400 (16)	3.6(11-8)	3.8(12-8)	4.1(13-8)	4.5(15-0)	6.2(20-6)
(2x8)	600 (24)	2.9(9-6)	3.1(10-4)	3.5(11-5)	3.9(12-10)	5.0(16-8)
	300 (12)	5.2(17-1)	5.6(18-5)	5.8(19-0)	6.4(21-1)	8.8(28-10)
50 x 250	400 (16)	4.5(14-10)	4.9(16-1)	5.3(17-4)	5.8(19-0)	7.7(25-4)
(2x8)	600 (24)	3.7(12-1)	4.0(13-1)	4.4(14.5)	4.9(16-3)	6.4(21-0)
	300 (12)	6.3(20-8)	6.8(22-4)	7.0(23-0)	7.7(25-4)	–
50 x 300	400 (16)	5.5(18-0)	5.9(19-6)	6/3(20-10)	7.0(23-0)	9.5(31-3)
(2x12)	600 (24)	4.4(14-7)	4.8(15-10)	5.3(17-6)	6.0(19-8)	7.8(25-6)
	300 (12)	3.7(12-3)	4.0(13-2)	4.0(13-2)	4.4(14-6)	6.4(20-11)
75 x 150	400 (16)	3.4(11-1)	3.6(12-0)	3.6(12-0)	4.0(13-2)	5.8(19-0)
(3x6)	600 (24)	2.7(9-1)	3.0(9-10)	3.3(10-10)	3.5(11-6)	4.8(16-0)
	300 (12)	5.0(16-4)	5.3(17-7)	5.3(17-7)	6.1(19-5)	8.4(28-0)
75 x 200	400 (16)	4.5(14-8)	4.8(15-11)	4.8(15-11)	5.3(17-7)	7.7(25-3)
(3x8)	600 (24)	3.7(12-1)	4.0(13-1)	4.2(14-0)	4.7(15-5)	6.5(21-3)

Table 2.825.1 (continued)

75 x 250 (3x10)	300 (12)	6.3(20-9)	6.8(22-3)	6.8(22-3)	7.5(24-6)	-
	400 (16)	5.7(18-9)	6.2(20-3)	6.2(20-3)	6.8(22-3)	-
	600 (24)	4.7(15-4)	5.0(16-8)	5.4(17-8)	5.9(19-6)	8.2(26-10)
100 x 150 (4x6)	300 (12)	4.1(13-8)	4.5(14-8)	4.5(14-8)	4.9(16-2)	7.1(23-4)
	400 (16)	3.7(12-4)	4.1(13-4)	4.1(13-4)	4.5(14-8)	6.4(21-2)
	600 (24)	3.2(10-8)	3.5(11-7)	3.5(11-8)	3.9(12-10)	5.6(18-7)
100 x 200 (4x8)	300 (12)	5.5(18-3)	5.9(19-7)	5.9(19-7)	6.6(21-8)	-
	400 (16)	5.0(16-7)	5.4(17-10)	5.4(17-10	5.9(19-7)	8.6(28-2)
	600 (24)	4.3(14-3)	4.7(15-6)	4.7(15-7)	5.3(17-7)	7.5(24-7)
100 x 250 (4x10)	300 (12)	7.1(23-5)	7.5(24-9)	7.5(24-9)	8.3(27-3)	—
	400 (16)	6.5(21-3)	7.0(22-11)	7.0(22-11)	7.5(24-8)	—
	600 (24)	5.5(17-11)	5.9(19-6)	6.1(20-0)	6.7(22-0)	—

Loadings given in the Table in pounds per square foot correspond to these combinations of design or working loadings:

3.2 KN/m^2 (67 psf)

Floor joists with plaster below: 2.4 kN/m^2 (50 psf) live load and 0.8 kN/m^2 (17 psf).

2.7 kN/m^2 (57 psf)

Floor joists with plaster below: 1.9 kN/m^2 (40 psf) live load and 0.8 kN/m^2 (17 psf) dead load; or roof rafters with a slope greater than 2 1/2 to 12: 1.4 kN/m^2 (30 psf) live load and 1.3 kN/m^2 (27 psf) dead load.

1.8 kN/m^2 (37 psf)

Ceiling joists over living rooms and usable attic space: 1.2 kN/m^2 (25 psf) live load and 0.6 kN/m^2 (12 psf) dead load; or Roof joists without plaster under: 1.4 kN/m^2 (30 psf) live load and 0.4 kN/m^2 (7 psf) dead load.

1.1 kN/m^2 (22 psf)

Ceiling joists without usable attic space: 0.5 kN/m^2 (10 psf) live load, and 0.6 kN/m^2 (12 psf) dead load.

The deflection of wood members shall not exceed L/360.

2.826 Minimum Construction Details Light-framed Construction

2.827 Columns or Posts

2.827.1 All wood columns and posts shall be framed to true-end bearings and shall be securely anchored against lateral or vertical forces.

2.827.2 All wood columns and posts shall be raised on plinths at least 200 mm above the ground or shall have the bottom protected by an effective moisture barrier.

2.827.3 Splicing of columns shall be done only in regions where lateral support is adequately provided about both axes.

2.827.4 No notching or cutting shall reduce the design dimensions of the column.

2.828 Studs

2.828.1 SIZE - Studs shall be not less than 50 mm x 100 mm nominal and where supporting more than one floor and a roof, shall be not less than 50 mm x 150 mm or 75 mm x 100 mm.

2.828.2 HEIGHT - Maximum allowable height of 50 mm x 100 mm and 75 mm x 100 mm stud framing shall be 4.25 mm and of 50 mm x 150 mm stud framing shall be 6 m unless the wall is otherwise laterally supported. Solid wood bridging shall be placed at intervals of not over 2.4 m. No studding shall be spaced more than 600 mm on centres unless vertical supporting members in the walls are designed as columns.

2.828.3 PLACING - Studs in exterior and bearing walls shall be placed with longest dimension perpendicular to the wall. Stud-bearing walls shall, so far as is practicable, be carried directly to the foundation or sills or beams at grade.

2.828.4 PLATES - The top plate of stud-bearing walls shall be doubled or lapped at each intersection with walls and partitions. Joints in the upper or lower members of the top plate shall be lapped not less than 1.2 m. Double plates shall be used around entire exterior walls.

2.828.5 BASE PLATES - Stud walls resting on masonry shall have base plates or sills of wood treated with an approved preservative. Sills of interior bearing walls, resting on masonry foundations walls where wood floor joists are to be used, and sills of exterior stud walls shall be of not less than 75 mm x 150 mm dimension, bolted to the masonry at the corners and at no more than 1.2 m intervals 12 mm bolts embedded 175 mm into the masonry or, in lieu thereof a 50 mm x 100 mm base plate, and each stud anchored past the base plate to the masonry with 3 mm x 25 mm steel strap or equivalent. Base plates of interior stud bearing walls resting on concrete slab floors shall be effectively fastened thereto, and such plates shall not be embedded in the concrete.

2.828.6 CORNERS AND BRACING - Corners of stud walls shall be framed solid by doubling the studs.

Bracing shall be provided of nominal 300 mm x 100 mm continous diagonal strips set into the face of thestuds and top and bottom plates at each corner of the building. Alternatively sheathing shall be of the standard described in clauses 2.832.9 to 2.832.12.

2.828.7 SPLICING - Bearings studs shall be spliced only at points where lateral support is provided .

2.828.8 NOTCHING - No notching or cutting whatsoever shall be permitted in studs which carry loads in excess of 75% of their capacity.

2.828.9 PIPES IN WALLS - Stud partitions containing plumbing or other pipes shall be so framed and the joists underneath so spaced as to give proper clearance for piping. Where a partition containing such piping runs parallel to the floor joists, the joists shall be doubled and spaced to permit the passage of such pipes and shall be bridged. Where plumbing or other pipes are placed in or partly in a partition necessitating the cutting of the plates, a metal tie not less than 3 mm thick and 40 mm wide shall be fastened to each side of the plate across the opening with 4-16d nails at each end of the each strap.

2.828.10 HEADERS - All openings in bearing walls 1.2 m or less in width shall be provided with headers of not less than 50 mm nominal thickness, placed on edge, and all openings more than 1.2 m wide shall be trussed or provided with headers or lintels. Such headers or tusses shall have not less than 50 mm nominal solid bearing at each end to the floor or bottom plate, unless other approved framing methods or joint devices are used.

2.828.11 STUD JOINING MASONRY - Where stud wall or partitions join masonry walls such studs shall be secured against lateral movement by nailing or bolting to the masonry.

2.828.12 INTERIOR PARTITONS - Interior partitions shall be constructed, framed and fire-stopped as specified for interior bearing walls, except that partitions may have a single top plate. In any occupancy, interior partitions not more than 1.2 m from a bearing wall and not exceeding 2.75 m in height may be of studs spaced 700 mm on centres and placed flat on the wall.

2.829 Firestops

2.829.1 Fire-resistance provisions shall be in accordance with Part 3 of this Code but in any case the following minimum provisions shall be made.

2.829.2 Fire-stopping shall be provided to cut off all concealed draft spaces both vertical and horizontal. Fire-stops shall form effectie fire barriers between storeys and between a storey and roof space. Fire-stops shall be provided in specific locations, as follows:

(a) Interior or exterior stud walls, at ceiling or floor levels.

(b) In all stud walls and partitions, so placed that the maximum dimensions of any concealed space is not over 2.4 m.

(c) Between stair stringers at intervals not exceeding 2.1 m of vertical height and at top and bottom.

(d) Around sliding door pockets.

(e) Other locations not specifically mentioned such as holes for pipes, shafting, behind furring strips and similar places which could afford a passage for flames.

(f) Fire-stops when of wood, shall be of 50 mm nominal thickness in direction of protection.

(g) Horizontal fire-stops of attic and ceiling plenums shall be provided.

2.830 Joists and Rafters

2.830.1 SIZE - The minimum of joists and rafters shall be as specified in sub-section 2.825 of this Section and shall be not less than 50 mm nominal size.

2.830.2 SPACING - Maximum spacing of joists and rafters, where a plaster ceiling is directly supported on the bottom of such members shall be 400 mm on centres, for other types of ceiling the maximum spacing will be 1.2 m.

2.830.3 BEARING - Joists and rafters shall bear on wood plates and shall not be directly in contact with masonry; except that joists and rafters, when more than 1.8 m above grade and bearing on concrete beams cast in masonry walls which extend above the wood joists and raftes, may bear on such concrete beams provided the ends shall be fire cut and anchored as specified in 2.830.4.

2.830.4 Joists and rafters shall not have less than 100 mm of bearing, except as follows:

(i) Ceiling joists may butt into the web of a steel beam and be neatly fitted to bear on not less than 75 mm wide bottom flange of such beam. Joists and rafters bearing on top of concrete beam and where not parapet is to be erected shall bear on a wood plate, secured to the concrete with 15 mm diameter bolts, 250 mm long. Where 3 mm x 25 mm anchors embedded in the concrete beam secure directly to the rafters or roof joists, such plate shall be not less than a 50 mm x 100 mm with bolts to the masonry not more than 1.2 m apart; and where anchorage is provided only from the plate to the rafter or roof joist such plate shall not be less than 75 mm x 100 mm with bolts to the masonry not more than 1.2 m apart.

(ii) Floor joists may butt into a header if effectively toe-nailed and if an approved saddle providing not less than 75 mm of bearing transmits the vertical load to the top of the header.

2.830.5 ANCHORAGE - Joists fire cut into a masonry wall shall be anchored to the concrete beam on which they bear. Such anchors shall be spaced not more than 1.2 m apart and shall be placed at opposite ends across the building on the same run of joists.

2.830.6 All joists shall be nailed to the bearing plates, to each other where they lap, and to the studs where such studs are adjacent; and ceiling joists shall be nailed to roof rafters, if practicable.

2.830.7 Every roof rafter and/or roof joist shall be anchored to the beam or studs on which they bear, and roof rafters opposing at a ridge shall be anchored across the ridge.

2.830.8 Anchors securing wood to concrete shall be of not less than 3 mm x 25 mm strap iron embedded in the concrete and nailed to the stud or joist or rafter with not less than 3/16d galvanised nails or shall be a commercial anchor approved by the relevant authority, anchoring each member to a plate provided such plate is not less than 50 mm x 100 mm and anchored to the concrete by bolts spaced not more than 1.2 m apart.

2.830.9 Anchors securing wood to wood shall be of 3 mm x 25 mm strap, nailed to each member with 3/16d galvanised nails, or shall be a commercial anchor approved by the relevant authority, anchoring each member.

2.830.10 Any anchoring systems shall be continuous from the foundations to the roof and shall satisfy the uplift requirements of Part 2, Section 2 - Wind Loads.

2.830.11 SPLICING - No horizontal members shall be spliced between points of support, except that the relevant authority in special cases may approve properly designed and bolted splices.

2.830.12 FLOOR JOISTS - Floor joists under all walls parallel to any joists shall be doubled. Such doubled joists may be separated not more than 125 mm by solid blocking spaced 1.2 m intervals.

2.830.13 CEILING JOISTS - In buildings without parapet walls, the ceiling joists, where practicable, shall be nailed to the rafters to act as a collar tie.

Ceiling joists shall not be used to support rafter loads.

2.830.14 ROOF JOISTS - Roof joists may cantilever over exterior walls as limited by the allowable stress, but the length of such cantilever shall not exceed the length of that portion of such joist inside the building.

2.830.15 ROOF RAFTERS - Hip rafters, valley rafters and ridges shall be required and shall be not less in size than the largest rafter framing thereto nor less than required to support the loads.

2.830.16 BRIDGING - Bridging of floor and roof joists shall be provided where necessary.

2.831 Suspended or Furred Ceilings

2.831.1 JOISTS OR FURRING - Joists or furring supporting a plaster ceiling shall be spaced not more than 400 mm on centres.

2.831.2 Joists of a suspended ceiling shall not be less than 50 mm x 100 mm members, and wood hangers shall provide nailing and be less than the equivalent of 25 mm x 100 mm.

2.831.3 Furring of a ceiling in contact with supporting joists shall not be less than 25 mm x 75 mm or 50 mm x 50 mm for spans of 1 m and for longer spans should be designed as joists.

2.832 Sheathing

2.832.1 FLOOR SHEATHING - Floor sheathing where part of a required fire-resistance assembly shall comply with the chapter on Fire-resistive Standards.

2.832.2 The finished floor shall be tongued and grooved not less than nominal 25 mm lumber laid perpendicular to thejoists with end joints on the joists, or a sub-floor shall be provided as set forth in the following sub-section.

2.832.3 Lumber sub-flooring shall be laid diagonally, shall be not less than 25 mm nominal wood, end joints shall be on joists, joints shall be staggered and parallel to the joists and ends at walls and similar places shall be supported by a ribbon or by blocking.

2.832.4 Flooring shall be nailed with 8d common nails not less than two in each board at each support.

2.832.5 Floors of mill-type buildings shall be sheathed as specified for mill floors.

2.832.6 Flooring shall not extend closer than 12 mm from masonry walls.

2.832.7 ROOF SHEATHING - Wood roof sheathing shall be tongued-and-grooved or shall be plywood except as may be otherwise approved by the relevant authority.

2.832.8 Tongued-and-grooved roof sheathing shall have thickness of not less than 20 mm without tolerance and sheathing of such thickness shall span not more than 700 mm between rafters or joists, shall have staggered joints and shall be nailed with 8d common nails not less than 50 mm each 150 mm board nor 75 mm each 200 mm board at each support.

2.832.9 STORM SHEATHING - Exterior stud walls shall be wind braced with storm sheathing for areas where expected wind speed is in excess of 112 kilometres per hour or in earthquake prone area.

2.832.10 Such storm sheathing shall be tightly fitted, diagonally placed, tongued-and-grooved sheathing, not less than 20 mm thickness without tolerance, nailed with two 8d common nails for 150 mm wide boards, and three 8d common nails for 200 mm wide boards to each support; or shall be the maximum practicable size plywood panel, not less than 8 mm in thickness, nailed with 6d common nails for 8 mm , 10 mm and 12 mm thickness and 8d common nails 16 mm thickness and all such nails shall be 150 mm on centres at edges of panel and 300 mm on centres to interior panel supports, for walls other than single lined exterior cladding.

2.832.11 An effective water barrier shall be provided under all wood exterior cladding between the cladding and the supporting studs, and all openings shall be flashed.

2.832.12 Where plywood less than 10 mm in thickness is used for stom sheathing, cladding suitable to resist the horizontal wind forces shall be applied over the storm sheathing.

2.832.13 All plywood permanently exposed in outdoor locations shall be of exterior type, and where used for roof or exterior wall sheathing shall meet the standards for exterior type plywood in Appendix B unless otherwise approved by the relevant authority.

2.832.14 All plywood used structurally shall bear the identification of the manufacturer as to type and grade, species of veneer used and appropriate commercial standard.

2.833 Furring

2.833.1 Where the interior of masonry walls are furred, such furring shall be treated in fire-stopped as herein required and shall be securely fastened to the masonry with not less than one out nail in alternate courses of block.

2.834 Connectors

2.834.1 The allowable loads on all types of connectors shall be as set forth in the standards listed in the appendices.

2.834.2 Number of nails required for connecting wood members is given in Table 2.834.1

TABLE 2.834.1
NAILING REQUIREMENTS

Connection	Common Wire Nails
Joist to Sill or Girder-Toe Nail	2-16d
Bridging to Joist-Toe Nail	2-8d
25 mm x 150 mm Sub-floor to Joist-Face Nail	2-8d
50 mm Sub-floor to Joist or Girder	2-16d
Plate to Joist or Blocking	16d-400 mm on centres
Stud to Plate-End Nail	2-16d
Stud to Plate-Toe Nail	3-16d or 4 8d
Top Plates-Spike Together	16d-600 mm on centres
Laps and Intersections	2-16d
Ceiling Joists-Toe Plate-Toe Nail	2-16d
Laps over Partitions	3-16d
to parallel rafters	3-16d
Rafter to Plate	3-16d
Continuous 25 mm Brace to Stud	2-8d
25 mm Sheathing to Bearing	2-8d
Corner Studs and Angles	16d-750 mm on centres
Plywood	8d-150 mm on centres edges 300 mm on centres interior
Anchors to Rafters or Roof Joists	3-16d

2.834.3 Nails, bolts and other metal connectors which are used in locations exposed to the weather shall be galvanised or otherwise protected against corrosion.

2.834.4 In general, nails shall penetrate the second member a distance equal to thickness of the membe being nailed thereto. There shall be not less than two nails in any connection.

2.834.5 Except for plywood and other laminated members manufactured under technical control and rigid inspection, gluing shall not be considered an acceptable connector in lieu of the connectors herein specified.

2.834.6 Safe loads and design practice for types of connectors not mentioned fully covered herein shall be determined by the relevant authority before approval.

2.835 Wood Supporting Masonry

2.835.1 No wood shall support masonry or concrete except as follows:

(a) Wood foundation piles may be used to support concrete or masonry.

(b) Wood joists may be used to support concrete and cement base tiles or terrazzo floor surfaces for bathrooms of less than 9.3 sq. m in area, concrete block walls and clay tile walls having slabs not more than two and one-half inches in thickness.

(c) Wood rafters may support concrete roof tile.

2.836 Construction Details for Heavy Timber Construction

2.837 General

2.837.1 Heavy timber construction is that type in which fire-resistance is attained by placing limitations on the minimum size, thickness or composition of all load-carrying wood members; by avoiding concealed spaces under floor or roofs; by using approved fastenings, construction details, and adhesives; and by providing the required degree of fire-resistance in exterior and interior walls.

2.837.2 The following provisions represent the additional requirements over and above those requird for light framed construction set out in sub-sections 2.826 to 2.834.

2.838 Heavy Timber Framing

2.838.1 COLUMNS - Wood columns may be sawn or glue-laminated and shall be not less than 200 mm nominal in depth when supporting roof loads.

2.838.2 Columns shall be continuous or directly superimpsed one above the other with no girders or bolsters between columns, throughout all storeys by means of reinforced concrete or metal caps with brackets, or shall be connected by properly designed steel or iron caps, with pintles and base plates, or by timber splice plates affixed to the columns by means of metal connectors housed within the contact faces or other approved methods.

2.838.3 FLOOR FRAMING - Beams, girders and joists may be sawn or glued-laminated and shall be not less than 100 mm nominal in depth.

2.838.4 Framed or glued-laminated arches which spring from grade or the floor line and support floor loads shall not be less than 200 mm nominal, in any dimension.

2.838.5 Framed timber trusses supporting floor loads shall have members of not less than 50 mm nominal dimension.

2.838.6 ROOF FRAMING - Beams, girders and joists may be sawn or glued-laminated and shall be not less than 50 mm nominal, in least dimension. Framed members or glued-laminated arches which spring from the floor line and do not support floor loads shall have members not less than 50 mm nominal in width and 150 mm nominal in depth for the lower half of height and not less than 50 mm nominal, in any dimension for the upper half of the height. Framed members or glued-laminated arches which spring from the top of the walls or wall abutments, framed timber trusses and other roof framing which does not support floor loads shall have members not less than 100 mm nominal in width and 150 mm nominal in depth.

2.838.7 CONSTRUCTION DETAILS - Wall plate boxes of self-releasing type or approved hangers shall be provided where beams and girders enter masonry. An air space of 25 mmm shall be provided at top, end, and sides of members unless approved durable or treated wood is used. Girders and beams shall be cross-tied by caps to transfer horizontal load across the joint. Wood bolsters may be placed on top of columns, which support roof loads only. Intermediate beams used to support floors shall rest on top of girders, or be supported on approved metal hangers which transmit the vertical load to the top of the girder. Columns, beams, girders, arches and trusses of material other than wood shall have a fire-resistive rating of not less than one hour. Wood beams and girders supported by masonry walls shall have not less than 100 mm of solid masonry between their ends and the outside face of the walls. Roof anchors shall be proved as set forth in this chapter but not less than required to resist the uplift loads set forth in Part 2 Section 1 on Live and Dead Loads.

2.838.8 CONCEALED SPACES - Floors and roof decks shall be without concealed spaces, except that building service equipment may be enclosed provided the spaces between the equipment and enclosures are fire-stopped or protected by other acceptable means.

2.838.9 HEAVY TIMBER FLOORS - Floors may be of sawn or glued-laminated plank, splined or tongued-and-grooved of not less than 50 mm nominal thickness, or square edged plank not less than 75 mm nominal thickness, well spiked together. Planks shall be laid so that a continuous line of joints will not occur except at points of support. Planks shall be covered with 25 mm nominal tongued-and-grooved flooring laid crosswise or diagonally. Planks and floor shall not extend closer than 12 mm to the wall to provide an expansion joint and such expansion joint shall be covered at top and bottom.

2.838.10 HEAVY TIMBER ROOF DECKS - Roof decks shall be sawn or glued-laminated, splined or tongued-and-grooved plank, not less than 50 mm nominal, thickness well spiked together or a double thickness of 25 mm nominal, tongued-and-grooved boards with staggered joints. Other types of decking may be used if non-combustible when approved by the relevant authority as being equal.

2.838.11 CAMBERING - Trusses and span girders shall be designed with sufficient camber, or other provision shall be made to counteract any possible deflection.

2.839 Protection of Wood

2.840 Preservative Treated or Durable Species Wood

2.840.1 All wood in areas where deterioriation would affect structural safety shall be treated in an approved method with an approved preservative or shall be of durable species as approved by the relevant authority.

2.840.2 All wood in contact with, or less than 450 mm from, the ground shall be treated in an approved method with an approved preservative or shall be of a durable species as approved by the authority.

2.840.3 All wood in contact with concrete or masonry including sills, sleepers, plates, posts, columns, beams, girders and furring shall be treated in an approved method with an approved preservative or shall be of durable species as approved by the relevant authority, except that the ends of joists not less than 2.4 m above grade when in contact with concrete or masonry, may be treated by dipping the ends in an approved preservative for a period of not less than five minutes.

2.840.4 Approval of the method and materials of treatment with a preservative shall be in accordance with the Standards set forth in Appendices.

2.841 Ventilation

2.841.1 Attic space between ceiling joists and roof rafters shall be effectively ventilated. Openings shall be located to provide effective cross ventilation, and such openings shall be covered with a corrosion-resistant mesh with openings not greater than 1.5 mm.

2.842 Light and Ventilation

2.842.1 The space between the bottom of wood floor joists and the ground of any building, except such space as occupied by a basement or cellar, shall have ventilating openings through foundation walls and such openings shall be covered with a corrosion-resistant wire mesh with openings not greater than 1.5 mm. Where practicable ventilating openings shall be arranged on three sides. The minimum total area of ventilating openings shall be 0.2 square metre for each 4.5 metres of exterior wall. Such openings need not be placed in the front of the building.

2.842.2 Where wood floor joists are used, there shall be not less than 460 mm distance between the bottom of such floor joists and the grade beneath.

2.842.3 Access shall be provided for all crawl spaces.

PART 2

STRUCTURAL DESIGN REQUIREMENTS

SECTION 8
STRUCTURAL TIMBER

APPENDIX A
SIZES OF STRUCTURAL LUMBER

APPENDIX A - SIZES OF STRUCTURAL LUMBER

Standard Sizes of Yard Lumber and Timbers

Details regarding the dressed sizes of various species of lumber are provided in the grading rules of the agencies which formulate the maintain such rules. The dressed sizes set forth in the American Softwood Lumber Standard, Voluntary Product Standard PS20-70. While these sizes are generally available on a commercial basis, it is good practice to consult the local lumber dealer to determine what sizes are on hand or can be readily secured.

TABLE A-1
NOMINAL AND MINIMUM - DRESSED SIZES
OF BOARDS, DIMENSION, AND TIMBERS

(The thicknesses apply to all widths and all widths to all thicknesses)

	Thicknesses			Face Widths		
		Minimum Dressed			Minimum Dressed	
Items	Nominal	Dry	Green	Nominal	Dry	Green
	mm	mm	mm	mm	mm	mm
Boards	25	19	20	50	37	39
	31	25	26	75	62	64
	37	31	32	100	87	89
				125	112	116
				150	137	141
				175	162	166
				200	181	187
				225	206	212
				250	231	237
				275	256	262
				300	281	287
				350	331	337
				400	381	387
Dimension	50	37	39	50	37	39
	62	50	52	75	62	64
	75	62	64	100	87	89
	87	75	77	125	112	116
	100	87	89	150	137	141
	112	100	102	200	181	187
				250	231	237
				300	281	287
				350	331	337
				400	381	387
Timbers	125 and thicker		12 off	125 and wider		12 off

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 8
STRUCTURAL TIMBER

APPENDIX B
LIST OF REFERENCES

APPENDIX B - LIST OF REFERENCES

1. National Design Specifications for Wood Construction - National Forest Products Association, Washington, D.C. 1982.

2. The Bahamas Building Code - Nassau, Bahamas.

3. Draft Seismic Regulations for Buildings - Applied Technology Council, U.S.A.

4. Structural Use of Timber - CCEO Draft Recommendations for CUBiC by Guyana Association of Professional Engineers.

5. American Lumber Standards for Softwood Lumber - NBS R16-53 U.S.A.

6. Structural Glued-Laminated Timber - Commercial Standard CS-253-63, NBS U.S.A.

7. Standard Specifications for Glued-Laminated Timber - American Institute of Timber Construction AITC 117 and 119.

8. Plywood Lumber Structural Assemblies Handbook, American Plywood Association U.S.A.

9. Southern Builing Code - Southern Building Code Congress, U.S.A.

10. "Timber" Guyana Timber Export Board - Georgetown, Guyana.

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 8
STRUCTURAL TIMBER

APPENDIX C
FIRE-RESISTANCE OF SOME CARIBBEAN TIMBERS

APPENDIX C - FIRE-RESISTANCE OF SOME CARIBBEAN TIMBERS*

Flame Penentration Test

Each sample of timber measured 400 mm x 200 mm x 25 mm with the grain parallel to the longest side. The 200 mm ends were cleated to prevent cracking as far as possible. The sample was placed vertically on its longest side, and the flame from a horizontally placed bunsen burner (using petrol gas) was so placed that the tip of the blue inner cone touched the surface of the wood. The gas pressure was kept as constant as possible, and the length of the blue cone dept at 30 mm. The temperature of the flame at the tip of the blue cone was approximately 1250^{o}C.

The time was taken from the moment of contact of flame and timber, to the moment when the opposite surface caught fire, the flame having burnt a hole through the wood.

VERY HIGH FIRE-RESISTANCE	Time of Penetration (minutes)
Pakuri	420
Purple Heart	420
Wallaba	360
HIGH FIRE-RESISTANCE	
Mora	292
Iron Wood	270
Pukardi	270
Kakaralli	250
MEDIUM FIRE-RESISTANCE	
Hububalli	218
Determa	204
Bullet Wood	203
Green Heart	192
Cabbage Palm	185
Kurahara	180
LOW FIRE-RESISTANCE	
Gale Silverballi	150
Washiba	145
Brown Silverballi	145
Trysil	139
Dauama	135

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* Extract from Draft "Use of Structural Timber in Buildings" November 1976 for CCEO by Guyana Association of Professional Engineers.

Kokotara	131
Tabanero	125
Crabwood	115
Dokooria	112
Keriti Silverballi	102

VERY LOW FIRE-RESISTANCE

Tauroniro	92
Morabukea	85
Tonka Bean	80
Douglas Fir	77
Crook	75
Baromalli	68
Pitch Pine	40

PART 2
STRUCTURAL DESIGN REQUIREMENTS

SECTION 8
STRUCTURAL TIMBER

APPENDIX D
TENTATIVE DATA ON CARIBBEAN TIMBERS

APPENDIX D - TENTATIVE DATA ON CARIBBEAN TIMBERS

The following tables provide merely a guide to some of the species of timber used in the Caribbean, with strength data indicating values encountered in the past.

For design purposes, Local Forest Products Laboratories should be consulted for authoritative recommendations.

These tables will be improved as more research is done or information obtained in this area.

TABLE D.1
NAMES AND DENSITIES OF SOME CARIBBEAN TIMBERS

Standard Name	Botanical Species	Other Common Names	Approximate Density (Seasoned) kgf/m^3
Caribbean Pine	Pinus Caribbea	Pine	608
Trinidad Teak	Tectona Grandis		530
Greenheart	Ocotea rodiaei	Sipiroe	977
Purpleheart	Peltogyne Pubescens	Amaranth	865
Kabukalli	Goupia giabra	Cupiuba	832
Manni	Symphonia Globulifera	Matakki	689
Mora	Mora excelsa	Nato	1,000
Shibidan	Aspidosperma Album	Red Peroba	850
Kurokai	Protium Decandrum	Gommier Rouge	641
Armomata	Clathrotropis Brachypetala	Acapu do Jgapo	1,000
Tatabu	Diplotropis Purpurea	Sucupira	1,000
Crabwood	Caropa guianensis	Andirdoa	610
Simarupa	Simaruba amara	Aceituno	430
Tauroniro	Humina Halsamilea	Bulletwood	900
Wamara	Swartzia Leiocalycina	Montouchi	1,100
Determa	Ocotea nubra	Wana	620
Hububalli	Loxopterygium	Koika	800

Red Cedar	Cedielaodocala	Cedar	480
Silverballi (Group)	Lauraceae	Pisie	270-1,000
Wallaba	Eperua Falcata	Bois	950
Yellow Wood	Podocarous Guatemalensis		
Banak	Virola Koschnivi		
Cabbage Bark	Tonchocarpus Castilloi		
Cramantee	Gurea Excelsa		
Mahogany	Swientenia Macrophylla		
Mayflower	Tabeuia Pentaphylla		
Nargusta	Termimalia Amazonia		
Santa Maria	Calophylum Brazilieneze Var		
Timbersweet	Nectrandra		
Waika Chewstick	Symphonia Globulifera		
Redwood	Tapirira Guianesis		
Yemeri	Vochvsia Hondurensis		

NOTE: 1. References for the above values are:

(a) "Timber" - Guyana Timber Export Board
(b) "Design Strengths of Belizean Woods" - Ministry of Works, Belize.

FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

$Feet^2$ (ft^2) x 0.0929030	= $Meters^2$ (m^2)
Acres x 4046.86	= $Meters^2$ (m^2)
$Miles^2$ (mi^2) x 2.58999	= $Kilometers^2$ (km^2)

VOLUME

$Feet^3$ (ft^3) x 0.02831685	= $Meters^3$ (m^3)
$Feet^3$ (ft^3) x 28.31685	= Liters**
$Yards^3$ (yd^3) x 0.764555	= $Meters^3$ (m^3)
Acre feet x 1233.48	= $Meters^3$ (m^3)
Gallons (gal) x 0.00454609	= $Meters^3$ (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

$Feet^3$ per second (ft^3/s) x 0.02831685	= $Meters^3$ per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= $Meters^3$ per second (m^3/s)
Acre-feet per day x 0.0142764	= $Meters^3$ per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= $Meters^3$ per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 (lb-s/ft^2) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

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* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

NOTES

CARIBBEAN UNIFORM BUILDING CODE

PART 3

OCCUPANCY, FIRE SAFETY

AND

PUBLIC HEALTH REQUIREMENTS

Caribbean Community Secretariat
Georgetown
Guyana

1985

PART 3

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FOREWORD

The Caribbean Uniform Building Code is basically a set of minimum requirements respecting the safety of buildings on the aspects of public health, fire protection and structural sufficiency. The primary purpose is the promotion of public safety through the application of appropriate uniform building standards throughout the Commonwealth Caribbean.

This part of the Code, Occupancy, Fire Safety and Public Health Requirments, contains the requirements with respect to health and fire safety which depend on the type of occupancy of a building and the use to which it is put. The first section contains interpretive material relating to occupancy classification and the more general features of fire protection. Sections which follow contain specific requirements relating to building size and occupancy and fire safety within floor areas, exit requirements, requirements for service spaces, health requirements, fire protection systems, requirements during construction and signs.

Standard tests specified by the National Fire Prevention Association of the U.S., British Standards Institution, the American Society for Testing and Materials, and the New Zealand Codes of Practice are referred to as the standard for this Code for specific items such as sound transmission, definition of non-combustibility, and surface and rate of spread of flame. For items such as storage of volatile flammable liquids, smoke and heat vents, sprinkler systems, projection rooms with cellulose nitrate film, and installation of mechanical systems the requirements are specified to be in accordance with the relevant codes of the aforementioned agencies.

NUMBERING SYSTEM

For this part of the Code the numbering system is as follows:

The first number indicates the part of the Code, the first digit in the second number indicates the Section in the Part, the second and third digits in the second number indicate the Subsection in the Section, and the third number indicates the article in the subsection. These are illustrated as follows:

2	Part 2
2.500	Part 2, Section 5
2.506	Part 2, Section 5, Subsection 6
2.506.3	Part 2, Section 5, Subsection 6, Article 3

ARRANGEMENT OF SECTIONS

CARIBBEAN UNIFORM BUILDING CODE

DEFINITIONS

Definitions of words and phrases used in those requirements that are not included in the list of definitions in this Part shall have the meanings which are commonly assigned to them in the context in which they are used in these requirements, taking into account the specialised use of terms with the various trades and professions to which the terminology applies. The following words and terms in these requirements have the following meanings:

Alteration	means a change or extension to any matter or thing or to any occupancy regulated by this Code.
Appliance	means a device to convert fuel into energy and includes all components, controls, wiring and piping required to be part of the device by the applicable standard referred to in this Code.
Attic or roof space	means the space between the roof and the ceiling of the top storey or between a dwarf wall and a sloping roof.
Building	means any structure used or intended for supporting or sheltering any use or occupancy.
Building area	means the greatest horizontal area of a building above grade within the outside surface of exterior walls or within the outside surface of exterior walls and the centre line of firewalls.
Building height (in storeys)	means the number of storeys contained between the roof and the floor of the first storey.
Closure	means a device or assembly for closing an opening through a fire separation, such as a door, a shutter, wired glass or glass block, and includes all components such as hardware, closing devices, frames and anchors.
Exhaust duct	means a duct through which air is conveyed from a room or space to the outdoors.
Exit	means that part of a means of egress that leads from the floor area it serves, including any doorway leading directly from a floor area, to a public thoroughfare or to an acceptable open space.
Exit, access to	means that part of a means of egress within a floor area that provides access to an exit serving the floor area.

Exit, horizontal means that type of exit connecting 2 floor areas at substantially the same level by means of a doorway, vestibule, bridge or balcony, such floor areas being located either in different buildings or located in the same building and fully separated from each other by a firewall.

Exit level means the lowest level in an enclosed exit stairway from which an exterior door provides access to a public thoroughfare or to an acceptable open space with access to a public thoroughfare at approximately the same level either directly or through a vestibule or exit corridor.

Exit storey means a storey from which an exterior door provdies direct access at approximately the same level to a public thoroughfare or to an acceptable open space with access to a public thoroughfare.

Fire compartment means an enclosed space in a building that is separated from all other parts of the building by enclosing construction providing a fire separation having a required fire-resistance rating.

Fire damper means a closure which consists of a normally held open damper installed in an air distribution system or in a wall or floor assembly, and designed to close automatically in the event of a fire in order to maintain the integrity of the fire separation.

Fire-protection rating means the time in hours or fraction thereof that a closure will withstand the passage of flame when exposed to fire under specified conditions of test and performance criteria, or as otherwise prescribed in this Code.

Fire-resistance means the property of a material or assembly to withstand fire or give protection from it; as applied to elements of buildings. It is characterized by the ability to confine a fire or to continue to perform a given structural function, or both.

Fire-resistance rating means the time in hours or fraction thereof that a material or assembly of materials will withstand the passage of flame and the transmission of heat when exposed to fire under specified conditions of test and performance criteria, or as determined by extension or interpretation of information derived therefrom as prescribed in this Code.

Fire separation means a construction assembly that acts as a barrier against the spread of fire and may not be required to have a fire-resistance rating or a fire-protection rating.

Fire stop means a draft-tight barrier within or between construction assemblies that acts to retard the passage of smoke and flame.

Firewall means a type of fire separation of noncombustible construction which sub-divides a building or separates adjoining buildings to resist the spread of fire and which has a fire-resistance rating as prescribed in this Code and has structural stability to remain intact under fire conditions for the required fire-rated time.

Flame-spread rating means an index or classification indicating the extent of spread-of-flame on the surface of a material or an assembly of materials as determined in a standard fire test as prescribed in this Code.

Floor area means the space on any storey of a building between exterior walls and required firewalls, including the space occupied by interior walls and partitions, but not including exits and vertical service spaces that pierce the storey.

Flue means an enclosed passageway for conveying flue gases.

Guard means a protective barrier around openings in floors or at the open sides of stairs, landings, balconies, mezzanines, galleries, raised walkways or other locations to prevent accidental falls from one level to another. Such barrier may or may not have openings through it.

Heat detector means a device for sensing an abnormally high air temperature or an abnormal rate of heat rise and automatically initiating a signal indicating this condition.

Heavy timber construction	means that type of combustible construction in which a degree of fire safety is attained by placing limitations on the sizes of wood structural members an on thickness and composition of wood floors and roofs by the avoidance of concealed spaces under floors and roofs.
Interconnected floor space	means super-imposed floor areas or parts of floor areas in which floor assemblies that are required to be fire separations are penetrated by openings that are not provided with closures.
Limiting distance	means the distance from an exposing building face to a property line, the centre line of a street, lane, public thoroughfare or an imaginary line between 2 buildings on the same property, measured at right angles to the exposing building face.
Means of egress	means a continous path of travel provided by a doorway, hallway, corridor, exterior passageway, balcony, lobby, stair, ramp or other egress facility or combination thereof, for the escape of persons from any point in a building, floor area, room or contained open space to a public thoroughfare or other acceptable open space. (Means of egress includes exits and access to exists).
Occupancy	means the use or intended use of a building or part thereof for the shelter or support of persons, animals or property.
Plenum	means a chamber forming part of an air duct system.
Smoke alarm	means a combined smoke detector and audible alarm device designed to sound an alarm within the room or suite in which it is located upon the detection of smoke within that room or suite.
Smoke detector	means a device for sensing the presence of visible or invisible particles produced by combustion, and automatically initiating a signal indicating this condition.
Sprinklered	(as applying to a building or part thereof) means that the building or part thereof is equipped with a system of automatic sprinklers.

Storey means that portion of a building which is situated between the top of any floor and the top of the floor next above it, and if there is no floor above it, that portion between the top of such floor and the ceiling above it.

Suite means a single room or series of rooms of complementary use, operated under a single tenancy, and includes dwelling units, individual guest rooms in motels, hotels, boarding houses, rooming houses and dormitories as well as individual stores and individual or complementary rooms for business and personal service occupancies.

Unsafe condition means any condition that could cause undue hazard to life, limb or health of any person authorized or expected to be on or about the premises.

ABBREVIATIONS OF NAMES OF ASSOCIATIONS

ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
BSI	British Standards Institution
NFiPA	National Fire Protection Association of the U.S.
UL	Underwriters' Laboratories (of Canada)

ABBREVIATIONS OF WORDS AND PHRASES

cm	centimetre(s)
cm^2	square centimetre(s)
CO_2	carbon dioxide
Cp	horizontal force factor
^oC	degree(s) Celsius
^oF	degree(s) Fahrenheit
G	acceleration due to gravity
kg	kilogram(s)
kPa	kilopascal(s)
l	litre(s)
kW	kilowatt(s)
lx	lux
m	metre(s)
m^2	square metre(s)
m^3	cubic metre(s)
max	maximum
min	minimum
mm	millimetre(s)
mPa	megapascal(s)
O.D.	outside diameter
t	tonne(s)

CARIBBEAN UNIFORM BUILDING CODE

PART 3

OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 1

OCCUPANCY AND CONSTRUCTION CLASSIFICATION

PART 3

SECTION 1

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TABLES

SECTION 1

3.100 OCCUPANCY AND CONSTRUCTION CLASSIFICATIONS

3.101 Scope

3.101.1 This section of the Caribbean Uniform Building Code deals with the Occupancy Classification of buildings, the Construction Classification (the fire ratings of various types of buildings) and Public Health.

3.101.2 All new or existing buildings whatever their size, or of whatever type of construction, must be placed within one of the categories set out below by the Building Control Authority. The Category into which the building is placed shall be determined by its use and throughout this Code, the category according to use shall be known as its Occupancy Group. In certain instances, for clarity, the words 'use group' may also occur; these terms shall be regarded as synonymous.

3.101.3 The section deals with aspects of, and requirements for the sub-sections as outlined in (a) to (x) below:

3.102 General Requirements

3.102.1 No building or structure shall be constructed nor shall any lot or portion of a lot be subdivided or sold nor any lot line moved by sale of land or otherwise in such a manner as to eliminate, nullify or reduce any required spaces for light and ventilations or for exit purposes or in any way to create violations or for exit purposes or in any way to create violations of any of the provisions of this Code.

3.103 Occupancy Classification

3.103.1 Every building or portion thereof whether existing or hereafter erected shall be classified by the Building Control Authority according to its use or the character of its occupancy as a building of Group A, B, F, H, I, M, R, S, and T Occupancy. When minor accessory uses do not occupy more than 10 percent of the area of any floor or a building, (not more than 10 percent of the basic area permitted by Occupancy), the major use of the Building shall determine the Occupancy classification.

3.103.2 Short list clarifying broad groupings:

A for Assembly (split up as below)	(see sub-section 3.104)
B for Business Occupancies	(see sub-section 3.105)
F for Factory Occupancies	(see sub-section 3.106)
H for High Hazard Occupancies	(see sub-section 3.107)
I for Institutional Occupancies (split up as below)	(see sub-section 3.108)
M for Mercantile Occupancies (split up as below)	(see sub-section 3.109)
R for Residential Occupancies (split up as below)	(see sub-section 3.110)
S for Storage Occupancies (split up as below)	(see sub-section 3.111)
T for Temporary and Miscellaneous Occupancies	(see sub-section 3.112)

3.103.3 Any Occupancy not specifically mentioned shall be classified by the buildings Control Authority in the Group it most nearly resembles.

3.103.4 FIRE GRADINGS OF BUILDINGS - All buildings whatever their occupancy group, shall be graded in accordance with the degree of fire hazard of their use in terms of hours and fractions of an hour and as covered under Section 6, (in Fire Resistive Construction Requirements).

3.104 Occupancy Group A - Assembly Buildings

3.104.1 GENERAL - All buildings and structures, or parts thereof, shall be classified in the assembly (A) use group which are used or designed for places of assembly as defined in this code. Assembly type uses with a total occupancy load less than fifty (50) shall be classified as use group B (Business).

3.104.2 OCCUPANCY GROUP A-1, THEATRES

3.104.3 OCCUPANCY GROUP A-1-A STRUCTURES - This group shall include all theatres and other buildings used primarily for theatrical or operatic performaces and exhibitions, arranged with a raised stage, proscenium curtain, fixed for portable scenery loft, lights, motion picture booth, mechanical appliances of other theatrical accessories and equipment, and provided with fixed seats.

3.104.4 OCCUPANCY GROUP A-1-B STRUCTURES - This group shall include all theatres without a stage and equipped with fixed seats used for motion picutre performances.

3.104.5 OCCUPANCY GROUP A-2 STRUCTURES - This group shall include all buildings and places of public assembly, without theatrical stage accessories, designed for use as dance halls, night clubs and for similar purposes including all rooms, lobbies and other spaces connected thereto with a common means of egress and entrance.

3.104.6 OCCUPANCY GROUP A-3 STRUCTURES - This group shall include all buildings with or without an auditorium in which persons assemble for amusement, entertainment or recreation, and incidental motion pictures, dramatic, theatrical or educational presentations, lectures, or other similar purposes without theatical stage other than a raised platform; and principally used without permanent seating facilities, including art galleries, exhibition halls, museums, lecture halls, libraries, restaurants other than night clubs, and recreation centres; and buildings designed for other similar assembly purposes including passenger terminals.

3.104.7 OCCUPANCY GROUP A-4 STRUCTURES - This group shall include all buildings used as churches, schools, colleges and for similar educational and religious purposes.

3.104.8 OCCUPANCY GROUP A-5 STRUCTURES - This group shall include grandstands, bleachers, coliseums, stadiums, drive-in theatres, tents and similar structures for outdoor assembly use, and shall simply with the provisions of this code for special uses and occupancies.

3.105 Occupancy Group B, Business Buildings

3.105.1 GENERAL - All buildings and stuctures or part thereof shall be classified in the business (B) Occupancy group which are used for the transaction of business or the rendering of professional services, or for other services that involve stocks of goods, wares or merchandise in limited quantities for use incidental to office uses or sample purposes; including among others offices, banks, civic administration activities, fire houses, police stations, professional services, testing and research laboratories, radio stations, telephone exchanges, and similar establishments.

3.106 Occupancy Group F, Factory and Industrial Buildings

3.106.1 GENERAL - All buildings and structures, or parts thereof, in which occupants are engaged in performing work or labour in fabricating, assembling, or processing of products or materials, shall be classified in the factory and industrial (F) occupancy group; including among others, factories, assembling plants, industrial laboratories and all other industrial and manufacturing uses, except those parts involving highly combustible, flammable or explosive products and materials of the high hazard group (occupancy group H).

3.106.2 LIST OF FACTORY AND INDUSTRIAL USES - The processes and manufacturers listed in the following Table 3.106.2 shall be indicative of, and include, the uses permitted in occupancy group F buildings.

TABLE 3.106.2

OCCUPANCY GROUP F, FACTORY AND INDUSTRIAL USES

Bakeries
Boiler works
Breweries
Canneries, including food products
Condensed and powdered milk manufacture
Dry cleaning using other than volatile flammable liquids in cleaning or dyeing operations
Electrical light plants and power houses
Electrolytic reducing works
Glass plants
Ice plants
Leather and tanneries, excluding enameling or japanning
Millwork and woodworking
Sugar refineries
Tenant factories, excluding ladies' dresses and other high hazard uses
Textile mills, including canvas, cotton cloth, bagging, burlap, carpets and rags
Upholstery and manufacturing shops
Water Pumping plants

3.106.3 SPECIAL INDUSTRIAL USES - All buildings and structures designed to house low hazard industrial processes, including, among others, the production and distribution of electrical, gas or steam power and rolling mills and foundries, requiring large areas and unusal heights to accommodate craneways or special machinery and equipment, shall be exempt from the height and area limitations (see under sub-section 3.206 General Area and Height Limitations).

3.106.4 CONSTRUCTION - Buildings and structures for such special industrial uses shall comply with the requirements of sub-section 3.114 except as to height, and when constructed of non-combustible (Type 2-C) construction may have balconies and mezzanine floors which do not exceed two-thirds (2/3) the area of the main floor in any one (1) tier.

3.106.5 EXTERIOR WALLS - The exterior walls of buildings of such low hazard industrial uses shall be constructed of approved non-combustible and weather-resisting materials and, when located with a fire separation of less than ten (10) metres from interior lot lines of any other building, shall be protected or constructed to provide a fire-resistance rating of not less than two (2) hours.

3.106.6 FIRE PROTECTION SYSTEMS - Special use industrial buildings herein defined shall comply with the requirements of Section 7 for fire protection systems; except that the provisions of sub-section 3.703 for automatic fire suppression systems in unlimited area buildings may be waived by the building official when such installations would be detrimental or dangerous to the specific use and occupancy.

3.107 Occupancy Group H, High Hazard Buildings

3.107.1 GENERAL - All buildings and structures, or parts thereof, shall be classified in the high hazard (H) occupancy group which are used for the storage, manufacture or processing of highly combustible or explosive products or materials, which are likely to burn with extreme rapidity, or which may produce poisonous fumes or explosions; for storage or manufacturing which involves highly corrosive, toxic or noxious alkalies, acids or other liquids or chemicals producing flame, fume, poisonous, irritant or corrosive gases; and for the storage or processing of any materials producing explosive mixtures or dust, or which result in the division of matter into fine particles subject to spontaneous ignition.

3.107.2 LIST OF HIGH HAZARD OCCUPANCIES - The processes, materials and manufactures listed in the following Table 3.107.2 are indicative of and shall be included among high hazard group.

TABLE 3.107.2

OCCUPANCY GROUP H, HIGH HAZARD USES

1. Acetylene gas and gases under pressure of fifteen 915) or more and in quantities of greater than seventy (70) cubic metres including hydrogen, illuminating, natural ammonia, chlorine, phosgene, sulphur dioxide, carbon monoxide, methyl oxide and all gases subject to explosion, fume or toxic hazard
2. Ammunition, explosives and fireworks manufacture
3. Apparel manufacturing
4. Artificial flowers and synthetic leather manufacture
5. Celluloid and cellúloid products
6. Cereal, feed, flour and grist mills
7. Cottom batting and cottom waste processes
8. Dry cleaning establishments using or storing more than fifteen (15) litres of gasoline or other hazardous liquids with a flash point under forty (40) degrees Celsius, or more than one hundred and fifty (150) litres of volatile inflammable liquids with a flash point between forty degrees Celsius ($40^{o}C$) and sixty degrees Celsius (60ºC) in a closed up tester
9. Fruit ripening processes
10. Grain elevators
11. Hydrogeneration processes
12. Industries employing solids or substances which ignite or produce flammable gases on contact with water
13. Kerosene, fuel, lubricating or any oil storage with a flash point under eighty degrees Celsius ($80^{o}C$)
14. Match manufacture or storage
15. Metal enamelling or japanning
16. Nitro-cellulose film exchanges and laboratories
17. Paint and varnish manufacture or spraying or dipping
18. Petroleum manufacture
19. Processing of paper or cardboard in loose form
20. Refrigerating systems using high hazard refrigerants
21. Shoe polish manufacture
22. Smoke houses (industrial)

Table 3.107.2 Cont'd

23. Straw goods manufacture or broom corn storage
24. Sugar and starch pulverizing mills
25. Tar, pitch or resin processing
26. Tanneries with enamelling or japanning
27. Tyre storage warehouses
28. Waste paper sorting or shredding, storage or baling

3.108 Occupancy Group I, Institutional Building

3.108.1 GENERAL - All Buildings and structures, or parts thereof, shall be classified in the institutional (I) occupancy group in which people suffering from physical limitations because of health or age are harboured for medical or other care or treatment, or in which people are detained for penal or correctional purposes, or in which the liberty of the inmates is restricted.

3.108.2 OCCUPANCY GROUP I-1 - This group shall include all buildings designed for the detention of people under restraint, including, among others, jails, prisons, reformatories, psychiatric units and similar uses.

3.108.3 OCCUPANCY GROUP I-2 - This group shall include all buildings used for housing people suffering from physical limitations because of health or age, including, among others, day nurseries, hospitals, sanitariums, clinics, infirmaries, orphanages, and homes for aged and infirm.

3.109 Occupancy Group M - Mercantile Buildings

3.109.1 GENERAL - All buildings and structures or parts thereof, shall be classified in the mercantile (M) occupancy group, which are used for display and sales purposes, including stocks of goods, wares, or merchandise incidental to such purposes and accessible to the public. These will include, among others, shops, salesrooms and markets, department stores, shopping malls or groups of shops. Any of the above that contain highly combustible goods, such as merchandise made from pyroxylin products shall be limited to small quantities that do not constitute a high hazard; if not so limited, construction shall comply with the requirements of the high hazard occupancy group given in sub-section 3.107.

3.110 Occupancy Group R - Residential Buildings

3.110.1 GENERAL - All buildings and structures or parts thereof, shall be classified in the residential (R) occupancy group in which families or households live, or in which sleeping accommodations are provided for individuals with or without dining facilities, excluding those that are classified as institutional buildings.

3.110.2 OCCUPANCY GROUP R-1 STRUCTURES - This group shall include all hotel and motel buildings, lodging houses, boarding houses and dormitory buildings arranged for the shelter and sleeping accommodation of more than twenty (20) individuals.

3.110.3 OCCUPANCY GROUP R-2 STRUCTURES - This group shall include all multiple-family dwellings having more than two (2) dwelling units; and shall also include all dormitories, boarding and lodging houses arranged for shelter and sleeping accommodation by more than five (5) and not more than twenty (20) individuals.

3.110.4 OCCUPANCY GROUP R-3 STRUCTURES - This group shall include all buildings arranged for the use of one or two-family dwelling units including not more than five (5) lodgers or boarders per family.

3.110.5 OCCUPANCY GROUP R-4 STRUCTURES - This group shall include all detached one or two-family dwellings not more than three (3) stories in height, and their accessory structures as indicated in Part 5, One and Two-Family Dwelling Code. All such structures may be designed in accordance with the One and Two-Family Dwelling Code or in accordance with the requirements for this Code for an Occupancy group R-3 Structure.

3.111 Occupancy Group S - Storage Buildings

3.111.1 GENERAL - All buildings and structures, or parts thereof, shall be classified in the Storage (S) occupancy group which are used primarily for the storage of goods, wares or merchandise, except those that involve highly combustible or explosive products or materials; including among others, warehouses, storehouses and freight depots.

3.111.2 OCCUPANCY GROUP S-1 STRUCTURES - This group shall include buildings used for the storage of moderate hazard contents, which are likely to burn with moderate rapidity, but which do not produce other poisonous gases, fumes or explosives, including, among others, the materials listed in the following Table 3.111.2 shall be classified in the S-1 storage occupancy group.

TABLE 3.111.2

OCCUPANCY GROUP S-1 STORAGE USES, MODERATE HAZARD

1. Bags, cloth burlap and paper
2. Bamboo and rattan baskets
3. Belting, canvas and leather
4. Books and paper in rolls or packs
5. Boots and shoes
6. Buttons, including cloth-covered, pearl or bone
7. Cardboard and cardboard boxes
8. Clothing, woollen wearing apparel
9. Cordage
10. Fibre board
11. Furniture
12. Furs
13. Glue, mucilagé, paste and size
14. Horn and combs, other than celluloid
15. Leather enamelling or japanning
16. Linoleum
17. Livestock shelters
18. Lumber yards
19. Motor vehicle repair shops
20. Petroleum warehouses for storage of lubricating oils with a flash point of one hundred and fifty degrees Celsius ($150^{\circ}C$) or higher
21. Photo-engraving
22. Public garages (Group 1) and stables
23. Silk
24. Soap
25. Sugar
26. Tobacco, cigars, cigarettes and snuff
27. Upholstering and mattress manufacturing
28. Wax candles

3.111.3 OCCUPANCY GROUP S-2 STRUCTURES - These buildings are those used for storage of non-combustible materials, and of low hazard wares that do not ordinarily burn rapidly. The following Table 3.111.3 lists some but not all of the materials that may be stored in buildings of the S-2 occupancy group.

TABLE 3.111.3
LOW HAZARD MATERIALS

1. Asbestos
2. Chalk and crayons
3. Food products
4. Glass
5. Metals
6. Motorcar spares (excluding upholstery)
7. Plumbing wares (metallic or ceramic pipe and fittings)
8. Porcelain and pottery
9. Talc and soapstones

3.112 Occupancy Group T - Temporary and Miscellaneous Uses

3.112.1 GENERAL - Structures and buildings of a temporary character and miscellaneous structures not classified in any other use or occupancy group, shall be constructed, equipped and maintained to meet the requirements of this Code commensurate with the fire and life hazard incidental to their use. Miscellaneous uses shall include all accessory buildings and/or structures used as private garages, builders and other sheds, reviewing stands, fences and similar purposes. It could also include such temporary shelters as tents and marquees and air supported structures.

3.112.2 DOUBTFUL OCCUPANCY CLASSIFICATIONS - When a building or structure is proposed for a use not specifically provided for under any of the occupancy groups in this Code, or when an existing building is to be used for some purposes where there is doubt about its new classification, the Building Authority or controlling official shall class it in the occupancy group it most nearly resembles in respect to the existing or proposed life and fire hazard.

3.113 Mixed Occupancies or Change in Use of Occupancy

3.113.1 TWO OR MORE USES - When a building is occupied for two or more uses i.e. where two or more occupancy classifications exist in the same building, this is quite acceptable provided the Building Control Authority has nominated those two or more occupancy classifications and that the divisions between the two or more differing occupancies, whether they be walls or floors, comply with the relevant fire code requirements as to the one and one-half or two hour fire ratings therein. (See under sub-section 3.114).

3.113.2 TWO OR MORE CODES GOVERNING THE TWO OR MORE OCCUPANICIES - When any such building having two or more occupancy groups therein has to comply with two or more codes of fire resistance, then the one that requires the highest rating shall be used for both or all occupancies i.e. when the code calls for one and one-half (1 1/2) hour fire resistance in party walls or floors, and the second or third occupancy classification calls for a two hour fire-resistance, then the whole building shall be constructed with all walls and floor having the longer - the two hour - rating.

3.113.3 INCIDENTAL USES - Where the higher or highest hazard use or occupancy classification is supplementary to the main use or classification of the building i.e. less than 10 percent of the total floor area of the building (see clause 3.113.1 above) then provided that such area of use is constructed and segregated by fire resistance rated construction as required under the relevant fire code, the building shall still be classified according to its main use and classification.

3.113.4 CHANGE IN OCCUPANCY CLASSIFICATION - No change shall be made in the occupancy of any building, existing, under construction, or new without the approval of the Building Control Authority.

3.113.5 Buildings in existence at the time of the introduction of this Code may have the existing use or occupancy continued, if such use or occupancy is legal at the time of the passage of this Code, provided such continued use is not dangerous to life. Nothing shall be construed to prohibit the Building Control Authority or its authorized representative from making inspections to see that the minimum standards of safety are maintained.

3.114 Construction Classification (Table 3.114 Follows sub-section 3.118)

3.114.1 GENERAL - All buildings and structures erected or to be erected, altered or extended in height or area shall be classified in any one, or in a combination of the four construction types herein defined:

(i)	Type 1	Fireproof construction (includes also Types 1-A and 1-B).
(ii)	Type 2	Non-Combustible construction including Types 2-A, 2-B and 2-C).
(iii)	Type 3	Exterior masonry wall construction (includes also Types 3-A, 3-B and 3-C).
(iv)	Type 4	Frame construction (and Types 4-A and 4-B).

3.114.2 FALSE DESIGNATION - A building shall not be designated as a given type of construction unless it conforms to the minimum requirements for that type; and it shall be unlawful to post or use or designate or advertise a building as of a given type of construction unless it complies with the minimum code requirements for that type.

NOTE - The following sub-sections 3.115, 3.116, 3.117 and 3.118 are given as a guide for fire ratings under Construction Classification (1, 2, 3 or 4) but that Section of the Code (Section 6) Fire-resistive Construction Requirements shall be used when designing all buildings of whatever occupancy.

3.115 Type 1 - Highly Fire-Resistant Construction

3.115.1 GENERAL - Buildings and structures of fireproof construction are those in which the walls, partitions, structural elements, floors, ceilings and roofs, and the exitways are constructed and protected with approved non-combustible materials to afford the fire-resistance rating specified in Table 3.114, except as otherwise specifically regulated by the provisions of Section 6. Fire-proof buildings shall be further classified as Types 1-A and 1-B. Fire-retardant-treated wood may be used as specified in Table 3.114.

3.116 Type 2 - Non-Combustible Construction

3.116.1 GENERAL - Buildings and structures of non-combustible construction are those in which the walls, partitions, structural elements, floors, ceilings, roofs, and the exitways are constructed of approved non-combustible materials meeting the fire-resistance rating requirements specified in Table 3.114, except as modified by the fire limit restrictions of non-combustible buildings, and shall be further classified as Types 2-A, 2-B and 2-C. Fire-retardant-treated wood may be used as specified in Table 3.114 and Section 6.

3.117 Type 3 - Exterior Masonry Wall Construction

3.117.1 GENERAL - Buildings and structures of exterior masonry wall construction are those in which the exterior, fire and party walls are constructed of masonry or other approved non-combustible materials, of the required fire-resistance rating and structural properties; and the floors,roofs, and interior framing are wholly or partly of wood or of metal or other approved construction; the fire and party walls are ground supported; except that girders and their supports carrying walls of masonry shall be protected to afford the same degree of fire-resistance rating of the walls supported thereon; and all structural elements have the required fire-resistance rating specified in Table 3.114.

3.117.2 TYPE 3-A - Buildings and structures of heavy timber construction are those in which fire-resistance rating is attained by placing limitations on the minimum sizes of wood structural members and on minimum thickness and composition of wood floor and roofs; by the avoidance, or by the proper protection by fire-stopping or other acceptable means, of concealed spaces under floors and roofs; by the use of approved fastenings, construction details, and adhesives for structural members; and by providing the required degree of fire-resistance rating in exterior and interior walls (see Section 6 for construction information and detail).

3.117.3 COLUMNS - Wood columns may be sawn or glue-laminated and shall be not less than 200 millimetres, nominal, in any dimension when supporting floor loads and not less than 150 millimetres, nominal, in width and not less than 200 millimetres, nominal, in depth when supporting roof and ceiling loads only.

3.117.4 FLOOR FRAMING - Beams and girders of wood may be sawn or glue-laminated and shall be not less than 150 millimetres nominal, in width and not less than 250 millimetres, nominal, in depth. Framed or glue-laminated arches which spring from the floor line and support floor loads shall be not less than 200 millimetres, nominal, in any dimension. Framed timber trusses supporting floor loads shall have members of not less than 200 millimetres, nominal, in any dimension.

3.117.5 ROOF FRAMING - Framed or glue-laminated arches for roof construction which spring from the floor line or from ground level and do not support floor loads shall have members not less than 150 millimetres, nominal, in width, and not less than 200 millimetres, nominal, in depth for the lower half of the height and not less than 150 millimetres, nominal, in depth for the upper half. Framed or glue-laminated arches for roof construction which spring from the top of walls or wall abutments, framed timber trusses, and other roof framing which do not support floor loads, shall have members not less than 100 millimetres, nominal, in width and not less than 150 millimetres, nominal, in depth. Spaced members may be composed of two (2) or more pieces not less than 75 millimetres, nominal, in thickness when blocked solidly throughout their intervening spaces or when such spaces are tightly closed by a continuous wood cover plate of not less than 50 millimetres, nominal in thickness, secured to the underside of the members. Splice plates shall be not less than 75 millimetres, nominal, in thickness. When protected by approved automatic sprinklers under the roof deck, framing members shall be not less than 75 millimetres, nominal, in width.

3.117.6 FLOORING - Floors shall be without concealed spaces and shall be of sawn or glue-laminated plank, splined, or tongue-and-groove of not less than 75 millimetres, nominal, in thickness covered with 25 millimetres, nominal, dimension tongue-and-groove flooring, laid crosswise or diagonally, or 12 millimetres plywood, or 12 millimetres particle board; or of planks not less than 100 millimetres, nominal, in which, set on edge close together and well spiked, and covered with 25 millimetres, nominal, dimension flooring, or 12 millimetres plywood, or 12 millimetres particle board.

3.117.7 ROOF DECKING - Roofs shall be without concealed spaces and roof decks shall be sawn or glue-laminated, splined or tongue-and-groove plank, not less than 50 millimetres, nominal, in thickness, 30 millimetres thick interior plywood (exterior glue), or of planks not less than 75 millimetres, nominal, in width, set on edge close together and laid as required for floors. Other types of deckings may be used, if providing equivalent fire rating and structural properties.

3.117.8 BEARING WALLS - Bearing portions of exterior and interior walls shall be of approved non-combustible material and shall have a fire-resistance rating of not less than two (2) hours.

3.117.9 NON-BEARING WALLS - Non-bearing portions of exterior walls shall be of approved non-combustible materials, except as otherwise noted and where a horizontal separation of less than six (6) metres is provided, non-bearing exterior walls shall have a fire-resistance rating of not less than two (2) hours. Where a horizontal separation of six (6) metres to nine (9) metres is provided, non-bearing exterior walls shall have a fire-resistance rating of not less than one (1) hour. Where a horizontal separation of nine (9) metres or more is provided, fire-resistance rating is not required. Where a horizontal separation of six (6) metres or more is provided, wood columns and arches conforming to heavy timber sizes may be used externally.

3.117.10 TYPE 3-B - Structures of Type 3-B (ordinary protected) shall include all exterior masonry wall buildings in which the interior structural elements are wholly or partly of fire-protected wood of not less than 50 millimetres nominal thickness, or of other approved protected combustible materials, or of metal protected and insulated to afford the fire-resistance rating specified in Table 3.114.

3.117.11 TYPE 3-C - Structure of Type 3-C (ordinary unprotected) construction shall include all exterior masonry wall buildings in which the interior structural members are of wood of not less than fifty (50) millimetres nominal thickness or consist of other combustible or non-combustible materials with protection of less than one (1) hour fire-resistance rating.

3.118 Type 4 - Frame Construction

3.118.1 GENERAL - Buildings and structures of frame construction are those in which the exterior walls, bearing walls, partitions, floor and roof construction are constructed wholly or partly of wood stud and joist assemblies with a minimum nominal dimension of fifty (50) millimetres, or of other approved combustible materials; with fire-stopping at all vertical and horizontal draft openings as regulated in Section 7, and in which the structural elements have the required fire-resistance ratings specified in Table 3.114. Frame buildings shall be further classified in Types 4-A and 4-B.

NOTES APPLICABLE TO TABLE 3.114

Note "a" For special high hazard occupancies involving a higher degree of fire severity and higher concentration of combustible contents, fire rating requirements for structural elements shall be increased accordingly.

Note "b" The fire separation or fire exposure in metres as herein limited, applies to the distance from other buildings on site or a boundary line or from across the street or other public space, not less than ten metres (10 m) to the building wall (or building line).

Note "c" Protected exteriors shall be required within the fire limits in Type 2 construction as follows:

H (high hazard) occupancy, two-hour fire resistance with separation up to four metres (4 m).

Note "d" See under Section 2 of this Code (General Building Limitations - Restrictions Outside Fire Limits).

Note "e" See under sub-section 3.113 (Mixed Occupancies or Change in Use of Occupancy).

Note "f" In all buildings of Types 3 or 4 construction, the stairways and their enclosures may be constructed of wood, or other approved materials of similar characteristics and of adequate strength. Exitways may be enclosed in one hour fire-resistance rated construction in buildings of three (3) stories or less in height.

Note "g" In Type 3-A construction, members which are of material other than heavy timber shall have a fire-resistance rating of not less than one hour (See also Section 2 General Building Limitations).

TABLE 3.114
FIRE RESISTANCE RATINGS (In Hours)

		TYPE OF CONSTRUCTION									
		Type 1 Sec 3.115		Type 2 Sec 3.116			Type 3 Sec 3.117			Type 4 Sec 3.118	
		Fireproof		Non-Combustible Note(c)			Exterior Masonry wall			Frame	
				Pro-tected		Unpro-tected	Heavy Timbers (Mill)	Ordinary Pro-tected	Ordinary Unpro-tected	Pro-tected	Unpro-tected
		1A	1B	2A	2B	2C	3A	3B	3C	4A	4B
Exterior walls (Note b)											
1. Fire Separation of 10m or more	Bearing	4	3	2	1	0	2	2	2	1	0
	NonBearing	0	0	0	0	0	0	0	0	0	0
1. Fire Separation less than 2m	Bearing	4	3	2	1½	1	2	2	2	1	1(d)
	NonBearing	2	2	1½	1	1	2	2	2	1	1(d)
1. Fire Separation more than 2m but less 4m	Bearing	4	3	2	1	0	2	2(c)	2	1	0
	NonBearing	2	2	1½	1	0	2	2	2	1	0
1. Fire Separation more than 4m but less 10m	Bearing	4	3	2	1	0	2	2	2	1	0
	NonBearing	1½	1½	1	1	0	Sec.3.117	1½	1½	1	0
2. Fire walls and party walls		4	3	2	2	2	2	2	2	2	2
3. Fire enclosure of exitways, halls, stairways (Note f)		2	2	2	2	2	2	2	2	1	1
4. Shafts other than exitways (elevator)etc.		2	2	2	2	2	2	2	2	1	1
5. Exitway access corridors (Note j)		1	1	1	1	1(h)	1(h)	1	1	1	1
6. Vertical separation of tenant spaces		1	1	1	1	0(h)	1(h)	1	0	1	0
7. Dwelling unit separations (Note k)		1	1	1	1	1(h)	1(h)	1	1	1	1
7. Other nonbearing partitions		0	0	0	0	0(h)	0(h)	0	0	0	0
8. Interior bearing walls & partitions, columns girders (not roof); and framing (Note 1)	Supp more than 1 fl	4	3	2	1	0	Sec.3.117	1	0	1	0
	" 1 fl. only	3	2	1½	1	0	"	1	0	1	0
	" roof only	3	2	1½	1	0	"	1	0	1	0
9. Structural member supporting wall (not less than fire-resistance rating wall supported)		3	2	1½	1	0	1	1	0	1	0
10. Floor construction incl. beams (Note g)		3	2	1½	1	0	Sec.3.117	1	0	1	0
11. Roof construction, beams trusses, fr. arches on roof deck (Notes g&i)	5m or less in hgt to lowest mbr	2	1½	1(h)	1(h)	0	"	1	0	1	0
	over 5m but less	1	1	1	0	0	"	0	0	1	0
	7m or more	0(m)	0(m)	0(h)	0(h)	0	"	0	0	0	0

Note "h" Fire-retardant-treated wood complying with Section 6 of this code may be used in Type 2 construction.

Note "i" Where the omission of fire protection from roof trusses, roof framing and decking is permitted, the horizontal or sloping roofs in Types 1 and 2 buildings immediately above such members shall be constructed of non-combustible materials of the required strength without a specified fire-resistance rating or of mill type construction in buildings not over five (5) stories in height (or twenty metres (20 m) in height - whichever is the lesser).

Note "j" Exterior exit corridors serving thirty (30) or fewer occupants may have a zero (0) fire-resistance rating (see Section 5 Means of Egress).

Note "k" Separation of all dwelling units shall have a fire-resistance rating of one hour.

Note "l" Interior bearing walls shall meet the requirements of Section 6 (Fire-resistive Construction Requirements) if serving a fire separation requirement.

Note "m" Buildings of H (high hazard), S-1 (moderate hazard storage), or M (Mercantile) Occupancies, when of Type I construction shall have not less than one hour fire-resistance rated roof construction (See also under Section 6 Fire-resistive Construction Requirements).

3.119 Party Walls (And Tenancy Separation Walls)

3.119.1 GENERAL - A party wall is one that is shared between the owner of two adjoining properties. It can be equally shared by being half on one property and half on a second i.e. where the actual boundary is at the centre of the wall. It can also still be a party wall if legally agreed (see below under 3.119.2) by both parties that the actual boundary is at one side or the other, of such a wall.

3.119.2 EXTERIOR WALLS

(A) Subject to the filing of a letter of permission to the Building Control Authority from the owner(s) of an existing adjoining building, the exterior walls thereof may be used as party walls when conforming to the following requirements:

(1) Where the type or types of construction used and/or combined floor areas of an existing and a proposed building are such that a separation into fire division is required, such walls shall meet the requirements for fire walls under this Code.

(B) Where two owners of adjoining properties before buildings are erected on either site arrange legally that the wall (or walls) between the buildings shall become party walls, they must file with the relevant Building Authority a copy of their legal agreement before permits to build are issued. This agreement must state:

(1) The occupancy group into which each building is to be classified;

(2) The type of construction (1, 2, 3, or 4) (See 3.114.1) each building will be constructed under; and

(3) The methods to be used by either owner - whichever is the first to build - by which the second owner can attach cross walls, beams, roof framing or other partitions to the legal party walls being erected by the first owner.

3.119.3 WALLS BETWEEN TENANTS - Walls between tenants whose occupancy groups are different from each other, including walls between tenants and exitways common to more than one tenant, shall have a fire rating of not less than one hour (Fire-resistive construction).

3.119.4 Where partial height partitions in dormitory or hostel accommodation (Occupancy R) are permitted under this Code, all such partitions shall be of non-combustible materials.

3.120 Hazardous Utilities

3.120.1 GENERAL - Individual feeders and shut-offs shall be provided for every separate fire division in every building.

3.120.2 Further requirements about hazardous materials and safety requirements for their use is covered under:

(a) Sub-section 3.107 Occupancy Group H - High hazard Buildings; and

(b) Section 6 Fire-Resistive Construction Requirements.

3.120.3 ELECTRICAL - Electrical installations shall comply in every detail with the requirements of Part 4 Section 2 of this Code.

3.120.4 GAS - Gas cylinders shall be located on the exterior of buildings in a conspicuous and accessible place. All pipe connections shall be fitted in accordance with the Plumbing Code.

3.120.5 OTHER - Other utilities which may constitute hazards shall, in general, be governed by the provisions of this Section and shall be subject to such additional requirements as specified by the appropriate authority.

3.121 Exit Facilities

3.121.1 GENERAL - All buildings of whatever occupancy group must comply with the Means of Egress section (Section 5).

3.121.2 Where two or more occupancies having exit widths based on different numbers of occupants, occur on the same floor and have common exits, the number of units required for each such occupancy shall be calculated separately, and the units of width combined and proportioned to two or more exits as required by travel distance limitations of the most restricted occupancy.

3.122 Location of Property

3.122.1 GENERAL - The type of construction classification (sub-section 3.114) under which any building of whatever occupancy classification is erected (or to be erected) will govern its location on the site. E.g. in Table 3.114, which gives the fire-resistive ratings in hours of the various types of buildings, it will be noted that separation distance between buildings is a major factor in increasing the fire-resistance rating.

3.122.2 The location of all buildings and/or structures shall conform to the requirements of the Planning Authority and to the protection of certain opening requirements of the Group of Occupancy in which such building is classified in this Code, according to the use or the character of the occupancy, whichever is the more restrictive.

3.123 Sanitation

3.123.1 GENERAL - It should be noted that all plans and specifications for new, altered, or added-to buildings must be submitted to the Health Authorities for approval and should any special requirements be requested by them, these special requirements shall take precedence over and/or become additional to the requirements called for in this Code. E.g. the requirements for toilets for a fever hospital, or a quarantine section of a hospital could well be more stringent than this Code calls for.

3.123.2 Toilet facilities consisting of water closets and wash basins (or wash troughs) with running water, shall be installed in all buildings of all types of occupancy hereinafter erected, altered or added to. (The following clauses 3.123.4 to 3.123.20 cover general requirements and Table 3.123.3 gives specific requirements).

3.123.3 Table 3.123.3 gives numbers of water closets, urinal stalls and wash basins required for various occupancies and shall be used as a guide by controlling authorities. As for wash hand basins, the Health Authority has the power to increase or amend the number of units given in this Table for certain occupancies (see clause 3.123.11).

3.123.4 WATER CLOSETS - Water closets for public and/or staff use shall be of an elongated type, shall be equipped with open front seats and shall be separated from the rest of the room or space and from each other by stalls of impervious materials. Such stalls shall be equipped with self-closing doors with indicator, or other approved type, bolts on inside only; doors shall have not less than a 150 millimetre space at both top and bottom for ventilation and cleaning purposes.

3.123.5 WALLS AND FLOORS - Walls, up to height of not less than 1250 millimetres, and floors of all public toilet rooms shall be surfaced with either tiles or other approved impervious material, and the angle between walls and floors shall be covered or have other approved waterproof but cleanable angle fillet of same or similar material.

3.123.6 VENTILATED SPACES - Toilet rooms connected to or adjacent to rooms where food is prepared, stored, or served to the public shall be separated therefrom by an intervening ventilated space with close-fitting doors. Such intervening ventilated space shall not be common to toilet rooms of both sexes.

3.123.7 Toilet rooms connected to public rooms or passage-ways shall have an intervening ventilated space and shall be arranged or screened to insure decency and privacy.

3.123.8 Public toilets shall bear signs plainly indicating for which sex such room or rooms are intended.

3.123.9 Required facilities in public buildings shall be available to employees and the public without charge.

3.123.10 WASH BASINS OR TROUGHS - For every water closet in any building there shall be one wash basin with running water or one "place" with tap at a washing trough. Washing troughs, if used, shall be of stainless steel or they shall be tiled or lined with an approved impervious material with proper falls to trap waste(s).

3.123.11 NOTE - The above requirement of one wash basin to one water closet shall be used as a general rule, but as set out in Table 3.123.3, this proportion may be varied for certain types of occupancy, and whether the toilets are for public use, staff use, or private use. It may also be overidden by Health Department rulings for certain occupancies e.g. I (Institutional) and F (Factory and Industrial).

3.123.12 URINALS - It should be noted from Table 3.123.3, that when urinals are used in men's toilet areas the number of water closets may be reduced. All urinals shall be of acid-resistant impervious materials and whether they be single earthenware stalls, or multiple wall stalls, shall have approved gratings and trapped wastes to discharge direct into lower drains. Stainless steel banks or urinals shall have a non-slip stop and base channel all in one piece, and tiles or other impervious materials used in construction of urinals shall have channels and non-slip treads of the same or similar materials.

3.123.13 SHOWERS - Showers for the use of staff in certain dirty industries shall have either a hot or cold water supply serving them as required by health authorities. The requirements for, and the number required for any one industry (or occupancy) shall be given by the Health Authorities when plans are being submitted to them and a ruling must be obtained and provisions made in plans for these showers, before a permit to proceed is issued.

3.123.14 The position whether in the building or in a separate staff block adjacent to the buildings or factory will also be the subject of a Health Authority ruling. E.g. in certain dangerous chemical industries, it is often necessary to have showers available in close proximity to where employees are working, while in other industries such as perhaps steel mills or carbon processing for tyre retreading, showers in a separate building are quite acceptable.

3.123.15 HAND DRYING FACILITIES - Where wash basins or troughs are installed it is prudent, and more hygienic to install some form of hand drying facilities. These shall be roller towels, towel packs (either paper or fabric) or blower/dryers for which the air is heated by electricity or by gas flame. The installation of hand drying facilities is not a definite requirement of this Code for all occupancies, but is a recommendation that they be considered for occupancies such as certain sections of groups A, B, F, I and R. Again the Health Authority has power to direct in this regard.

3.123.16 LADIES' REST ROOMS - In certain occupancies (or Industries) where female labour is employed, this Code requires that rooms be set aside as ladies' rest rooms. It is preferable that these areas or rooms be attached to, or adjacent to toilets, but when this is not possible, then rest rooms shall have at least one water closet and one wash basin in close proximity.

TABLE 3.123.3

SANITATION FACILITIES
A GUIDE FOR CONTROLLING AUTHORITIES

NO. OF OCCUPANTS	WATER CLOSETS	URINALS	WASH BASINS	REST ROOMS	TYPES OF OCCUPANCIES
Up to 4 persons (male)		1	1	1	All
Up to 4 persons (female)	1		1		All
Up to 10 persons (mixed)	1		1		R3,R34 Some R1
Up to 12 persons (male)	1	1	1		H,I1*,I2*,R2
Up to 12 persons (female)	1		1	1	H,I1*,I2*, R2
Up to 20 persons (male)	1	1	1		F, H
Up to 20 persons (female)	2		1	1	F, H
Up to 25 persons (male)	1	1	1		F, H, M
Up to 25 persons (female)	2		2	1	F, H, M
Up to 50 persons (male)	2	2	3		F, M
Up to 50 persons (female)	3		3	1	F, M
50 - 100 persons (male)	4	4	5		B, M
50 - 100 persons (female)	5		5	1	B, M
Over 100 persons (male)	1/25 persons	1/25 persons	1/20 persons		B, H, Some T
Over 100 persons (female)	1/20 persons		1/20 persons	1+	B, H, Some T
Over 100 persons (male)	1/40 persons	1/40 persons	1/35 persons		A3, A4
Over 100 persons (female)	1/35 persons		1/35 persons	1+	A3, A4
Over 100 persons (male)	1/80 persons	1/80 persons	1/50 persons		A1,A1B,A2,A5
Over 100 persons (female)	1/50 persons		1/50 persons	1+	A1,A1B,A2,A5

*1 Occupancies shall be regarded as special cases and Health Authority will direct number of units.

3.123.17 SIZE: Although this Code does not state a size for each case, it is recommended that for a female staff of eight (8) persons or more, a room or space large enough for one couch and one chair be provided. For forty (40) female staff or more, the space should be large enough for two (2) couches and two (2) chairs separated by a draw curtain or screen. For larger numbers e.g. a clothing factory with 100 or more female workers, it is recommended that a proper sick bay with a nurse or attendant be provided and preferably in conjunction with a first aid room (see 3.123.20).

3.123.18 FIRST AID PROVISIONS

3.123.19 FIRST AID KITS OR BOXES - In all cases where people gather, in buildings of whatever occupancy group there is a code requirement that some responsible nominated person have charge of a first aid kit or red cross cabinet or box for use in minor emergencies.

3.123.20 FIRST AID ROOMS OR SICK-BAY - In buildings of any occupancy group, where the number of employees or work people regularly employed exceed 100, this code requires that a sick-bay or first aid room be provided. Where the greater proportion of workers or staff are women, a part or full time attendant is also required. The sick-bay or first aid room, shall have at least one couch, one chair, and one washbasin and be adjacent to or attached to a toilet. (See under Ladies' Rest Rooms - 3.123.10 - 3.123.17 above, for how a sick-bay for women staff or employees can be combined with a ladies' rest room).

3.124 Location of Toilets and/or Toilet Rooms

3.124.1 BUILDINGS OF TWO STORIES:

(a) Buildings of not more than two stories in height are required to have toilets for both sexes on each floor when there is more than one tenancy involved, or when toilets are likely to be used by the public.

(b) If both floors are occupied by the same tenant and the toilets are not for public use, but are for the use of tenants and staff only, it is permissible under this code to have men's toilets on one floor and ladies toilets on the other.

(c) For multi-storey buildings, e.g. office blocks, this code will allow for alternate toilet blocks (for men and women) on half-way landings of stairs, i.e. mezzanines. It is imperative that there be not more than a half-flight of stairs up or down to either men's or women's toilet rooms from any floor of the building.

(d) Toilets for public use in occupancies such as A-1 - A-5 and some in M, must have toilets for each sex on each floor. E.g. bars, restaurants, transportation terminals and similar.

3.125 Toilet Facilities For Physically Handicapped Persons

3.125.1 GENERAL

Any public buildings of whatever occupancy group to be or being erected, or any buildings that are altered to become a purely public building - e.g. post offices, pension offices, some bank buildings and public passenger terminals - shall have provisions incorporated for the physically handicapped and more especially for wheelchairs. (See also under Section 5 for access to the above type of buildings).

3.125.2 SIGNS

(a) All buildings erected which are required to (and do) comply fully with this Code shall have a plaque of not less than 150 millimetres square affixed in a prominent position at the main entrance bearing the international sign for the physically handicapped - (a stylized person in wheelchair - reproduced below).

(b) All toilet compartments, in any building so erected that comply with this Code, shall also bear on the door to that compartment a similar sign but the size of sign shall be not less than 50 millimetres square.

3.125.3 REQUIREMENTS

For the detailed requirements for the access for the disabled to buildings which include access to toilets and other rooms, recommendations on heights of handrails, grabrails, toilet and other fixtures, widths of passages and doors and other necessary information, designers and builders should use British Standard CP 96; New Zealand Code of Practice No. 4121; or Supplement No. 5 to the National Building Code of Canada.

3.126 Public Health Requirements

3.126.1 GENERAL

This section deals wih health standards that must be considered as necessary in all buildings, public or private, and of whatever occupancy in which they may be classified. It does not cover all aspects of public health as these are specifically set out in the Health Regulations which in turn are made enforcible by various Public Health Acts, but are meant as a guide to designers, builders and Building Authorities.

3.126.2 FOOD SCREENING

All areas where food is stored, packed or repacked, or prepared for human consumption, shall have all outside openings screened and all access doors equipped with second screened doors with self-closing devices attached. Screening shall be carried out to the approval of the Building and Health Control officers and shall be constructed of insect-proof wire mesh, preferably of some non-ferrous metal.

3.126.3 NOTE: "Food" in 3.126.2 above does not necessarily include prepacked foods in water and/or airtight containers or vegetables, but should include most fruits either fresh or dried. Storage of meat, fresh or frozen, and all meat products, are specifically covered in the Health Regulations and these regulations must be meticulously adhered to in all respects.

3.126.4 VERMIN PROOFING

Where food, fruit or vegetables are stored (other than in transit when it is only likely to be left in the one area for short periods) and these items are not stored in properly constructed freezers or cool rooms, the storage areas must be vermin-proof. Preferably storage areas should be constructed in brick, stone, concrete or concrete block on concrete floors, but if other methods of construction are used, then floors and wall linings shall be of an approved rodent-proof material. A water supply with standpipe(s) and tap(s) with hose connections shall be available either inside or within two metres of the access door to enable hosing/scrubbing/washing out of stores, and floors shall have either floor wastes built in, or floor sloped to a drain outlet which must be equipped with a rodent-proof non-ferrous grille.

3.126.5 CEILING HEIGHTS

The minimum ceiling height for all areas of human occupancy in buildings shall be 2.4 metres, i.e. the clear vertical distance between floor surface and underside of ceiling.

3.126.6 EXCEPTIONS

Although this is a general health requirement for buildings in all occupancy groups, there could be exceptions such as (a) a small room in a fully air-conditioned office building that is only occupied part time, or (b) a living room in a private residence with a sloping ceiling (provided the lowest point is not below 2.1 metres and the area with 2.4 metres or more in height is at least two thirds of the room area).

CARIBBEAN UNIFORM BUILDING CODE

PART 3
OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 2

GENERAL BUILDING LIMITATIONS

PART 3

SECTION 2

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PART 3

SECTION 2

3.200 GENERAL BUILDING LIMITATIONS

3.201 Scope

3.201.1 This section of the Caribbean Uniform Building Code is to define limitations of height and areas of buildings hereinafter erected, added to or altered, in certain fire limits or zones throughout the Caribbean as affected by the fire and life hazard incident to type of construction, occupancy groups, density of development, exterior exposure and accessibility of buildings to fire-fighting facilities and equipment.

3.201.2 This section deals with aspects of and requirements for the following sub-sections as outlined in (a) to (o) below:

3.202 Fire Limits/Zones

3.202.1 GENERAL - For the purpose of control of use and construction of buildings to prevent conflagration from fire, the Local Building Authority establishes limiting districts Designated "fire limits" and "outside fire limits", under the legal procedure of the jurisdiction for creating and establishing fire limits.

3.202.2 FIRE LIMITS - The fire limits or zones shall comprise the areas containing congested business, commercial manufacturing and industrial uses or in which such uses are developing.

3.202.3 OUTSIDE FIRE LIMITS - All other areas not included in the fire limits shall be designated as outside fire limits.

3.202.4 LOCAL AUTHORITY BY-LAWS - Each Local Authority in the Caribbean Countries adopting this Caribbean Uniform Building Code shall, if necessary add to their existing, or adopt a new by-law, preferably in conjunction with any existing building regulations, setting out the various fire zones or fire limits in their jurisdiction. From this by-law it will then be possible for designers, engineers, architects and others to submit detailed plans and specifications for the type of buildings allowed to be erected in such zones, all as required under this section of the Caribbean Uniform Building Code.

3.203 Restrictions Within The Fire Zones/Limits

3.203.1 GENERAL - All buildings, and additions to buildings hereinafter erected within the boundaries of the fire limits or zones shall be of fireproof (type 1); protected non-combustible, (types 2A and 2B); heavy timber (type 3A) or ordinary protected (type 3B), construction as defined in Part 3 Section 1 of this Code - see also Table 3.114 in that section for classification of the various fire-resistance ratings.

3.203.2 Buildings shall also be constructed within the height and area limitations set out in sub-section 3.206 below. Open Parking Buildings are covered separately in Part 3 Section 3 of this Code (3.331 and Table 3.331).

3.203.3 CERTAIN CONSTRUCTION TYPES PERMITTED - New buildings and additions to existing buildings hereafter erected within the fire limits or zones may be:

(a) Of unprotected non-combustible (Type 2C)

(b) Ordinary unprotected (Type 3C) or

(c) Protected frame (Type 4A) construction as defined in Part 3 Section 1 of this code and also as set out in Table 3.114 when constructed and located in accordance with the requirements of Table 3.203 on the next page.

TABLE 3.203

EXTERIOR WALL FIRE-RESISTANCE RATINGS REQUIREMENTS

Width of fire separation adjacent to exterior wall	Fire-resistance rating of exterior wall* or barrier	Fire-Resistance rating of exterior opening protectives	Classification of roof covering
On boundaries or less than 1 m therefrom any other building	4 hour	not permitted	B
More than 1 m but less than 2 m	3 hour	3 hour	B
2 metres but less than 3.5 metres	2 hour	1 1/2 hour	B
3.5 m or more but less than 9 m	1 hour	3/4 hour	B
9 m or more	0 hours	0 hours	C

* Not less than required by Table 3.114 in Part 3 Section 1 of Code.

NOTE 1 - The exterior wall or barrier shall extend to the full height of the building and be so constructed that it will remain structurally in place for the duration of the time indicated by the required fire-resistance rating.

NOTE 2 - When the exterior wall or barrier is adjacent to a flat roof, it shall be constructed with parapet.

3.203.4 STORM ENCLOSURES - PROTECTIVE PORCHES - Such enclosures may be erected of frame (Type 4) construction not more than 3 metres in height and not more than one metre wider than the entrance doors which they serve, provided they do not project more than 2 metres beyond the building line.

3.203.5 BUILDERS' SHEDS, TOOL STORES OR REVIEWING STANDS - Temporary builders' sheds erected in connection with approved building operations, platforms, reviewing stands, and other similar miscellaneous structures may be erected of frame (Type 4) construction for a limited period of time as approved by the Local Building Authority (See also 3.214 below - Temporary Structures).

3.203.6 BINS, TANKS, TOWERS, ROOF STRUCTURES AND RACKS:

3.203.7 TIMBER CONSTRUCTION - Coal and other storage bins, water towers, tankstand or trestles may be erected of mill type heavy timber construction, with dimensions not less than required for Type 3-A construction, provided they are not over 11 metres in height and located not closer to the lot boundary or any other building than 9 metres. When such structure is located on boundary lines along a waterfront or railway right of way, the above requirement is not applicable.

3.203.8 STRUCTURES AND ERECTIONS ON ROOFS OF BUILDINGS - Aerial supports not more than 4 metres in height, water tanks, and stands for same, stands for cooling systems (air conditioning) and flag poles may be erected of wood on buildings - not more than three (3) stories nor more than 12 metres in height, and drip bars in cooling towers may be constructed of wood.

3.203.9 MOTOR FUEL SERVICE STATIONS - Gasoline stations, and structures of similar business uses, not including high hazard (H) uses, may be erected of unprotected non-combustible (Type 2-C) construction within the height and area limits of occupation group B, provided they are located not less than 3.5 metres from the boundary line or from any other building. (See under 3.206 below, Table 3.207).

3.203.10 BUS AND PASSENGER TERMINALS - Roofs over parking lots and bus and passenger terminals may be erectd one (1) storey and not over 6 metres in height and not more than one thousand (1,000) square metres in area of non-combustible (Type 2-C) construction or of heavy timber mill (Type 3-A) construction.

3.203.11 SHOP OR STORE FRONTS - Wood veneers of 25 mm nominal thickness or exterior grade plywood not less than ten millimetres thick may be used on store fronts when facing public streets; provided the veneer does not exceed one (1) storey in height and is applied to non-combustible backing or is furred not to exceed 40 millimetres and fire stopped. (See also Part 3 Section 6, Fire-resistive Construction Requirements). Where all wood veneers comply with the relevant standard for exterior use, the height may be increased to two (2) stories.

3.204 Restrictions Outside Fire Limits

3.204.1 GENERAL - Outside the fire limits, all types of construction except as herein specifically prohibited, or for which special approval is required in connection with high hazard uses and occupancies in Part 3 Section 1, shall be permitted within the height and area limitations of Table 3.206 below.

3.204.2 BOUNDARY LINE - BUILDING LINE DISTANCE - In frame construction an exterior wall erected less than 2 metres (2 m) from its adjacent boundary shall be on one hour fire-resistance rated construction, except store fronts, and window and door openings in one and two-family dwellings. Exterior walls of Type 4 frame construction shall not be closer than one metre from any interior boundary lines.

3.204.3 ROOF COVERINGS - Roof coverings shall conform to the fire-resistive requirements of Class A, B, C or non-rated roofings complying with the provisions of Part 3 Section 6 (Fire-resistance Tests and Roof Coverings).

3.205 Existing Buildings

3.205.1 ALTERATIONS

3.205.2 LIMITATIONS - These provisions shall not be deemed to prohibit alterations within the limitations of Part 1 Section 1 of this Code (existing structures) provided no unlawful change of use/occupancy is involved.

3.205.3 MINOR CHANGES - Changes, alterations or repairs to the interior of a building and to the front facing a street or other public space may be permitted provided such changes, in the opinion of the Local Authority, do not increase the size or the fire hazard of the building, or endanger the public safety, and are not specifically prohibited by this Code.

3.205.4 EXISTING PROJECTIONS - A change or enlargement shall not be made to an existing part of a building now projecting beyond the street-boundary line or building line where such is established by law, except in conformity to the provisions of sub-section 3.211 governing new construction.

3.205.5 INCREASE IN HEIGHT AND AREA - It shall be unlawful to increase the height or area of an existing building or structure, unless it is of a type of construction permitted for new buildings of the increased height and area, and of occupancy or use group within the fire limit in which it is located and as regulated by Table 3.206 below.

3.206 General Area and Height Limitations

3.206.1 GENERAL - The areas and heights of all buildings and structures between exterior walls, or between exterior walls and fire walls, shall be governed by the type of construction and the occupancy group classification as defined in Part 3 Section 1 and shall not exceed the limits fixed in Table 3.206 below except as these may be specifically modified by other provisions of this Code.

3.206.2 AREA LIMIT - The area limitations specified in Table 3.206 below shall apply to all buildings fronting on a street, or public space not less than ten metres in width accessible to a public street.

3.206.3 HEIGHT LIMIT - The height in metres, and number of stories specified in Table 3.206 below shall apply to all buildings, and to all separate parts of a building enclosed within lawful fire walls complying with the provisions of Part 3 Section 6 (Fire-resistive Construction Requirements).

3.206.4 MULTI-STOREY BUILDINGS - Buildings two stories in height may be built to the same area limits provided in Table 3.206 for one-storey buildings. In buildings over two stories in height, the area limits of Table 3.206 for one-storey buildings shall be reduced as specified in the next Table 3.207 (see also next sub-section 3.207).

Notes applicable to Table 3.206 (see next page).

Note a. See the following clauses for general exceptions to Table 3.206

Clause 3.206.4 Allowable area reduction for multi-storey buildings and Table 3.207 following.

Clause 3.207.2 Allowable area increase due to automatic fire suppression system installation.

Clause 3.208.1 Unlimited area one-storey buildings.

Clause 3.209.1 Allowable height increase due to automatic fire suppression system installation.

NOTE b. Type 1 buildings permitted unlimited heights and areas are not subject to special requirements that allow increased heights and areas for other types of construction.

NOTE c. The tabular area of one-storey school buildings of use group A-4 may be increased two hundred (200) per cent provided every classroom has at least one (1) door opening directly to the exterior of the building. Not less than one-half (1/2) of the required exitways from any assembly room included in such buildings shall also open directly to the exterior of the building.

NOTE d. Church auditoriums of Type 3-A construction may be erected to 20 metres in height and of Type 4 construction to 14 metres in height.

NOTE e. For exceptions to height and area limitations of high hazard use buildings, see Part 3 Section 1 governing the specific use. For other special fire-resistive requirements governing specific uses, see Section 3.606.

NOTE f. For exceptions to height of multi-family dwellings of Types 2-B and 3-B construction, see Part 3 Section 6.

NOTE g. for height and area exceptions covering open parking structures, see Part 3 Section 3.

NOTE h. For height and area exceptions covering petroleum bulk-storage buildings, see Part 3 Section 6.

3.207 Area Exceptions

3.207.1 GENERAL - The provisions of this sub-section shall modify the area limitations/limits given in Table 3.206. There could be other special cases for special areas where this Code is adopted and these may be agreed to by dispensations from the Local Authority responsible.

3.207.2 AUTOMATIC FIRE SUPPRESSION SYSTEM - When a building of other than high hazard (occupancy group H) use is equipped with an approved automatic fire suppression system, the tabulated areas may be increased by two hundred (200) percent for one (1) storey buldings and one hundred (100) percent for buildings more than one storey in height.

3.207.3 SCHOOL BUILDINGS - When every classroom of one (1) storey school building (occupancy group A-4) has at least one (1) door opening directly to the exterior of the building, the tabulated area of Table 3.206 may be increased two hundred (200) percent. Not less than one-half (1/2) of the required exitways from any assembly room included in such buildings shall also open directly to the exterior of the building.

3.207.4 REDUCTION IN AREAS (MULTI-STOREY BUILDINGS) - As specified under clause 3.206.4 above, buildings of 2 stories in height may be built to the same area limits given in Table 3.206 for one-storey buildings, but in buildings over 2 stories in height, the area limits of Table 3.206 for one-storey buildings shall be reduced as specified in the following Table 3.207.

TABLE 3.206
HEIGHT AND AREA LIMITATIONS OF BUILDINGS

(height limitations in (a) Stories (above ground level); (b) metres; (c) (below) in are (m^2) per floor for buildings facing street 10 m or more wide or a Public space more than 10 m wide – see also "notes" before table a to h)

E.g. 6s.23m
$1340m^2$ means 6 stories, 23 metres in height, 1340 square metres per floor

TYPE OF CONSTRUCTION

OCCUPANCY GROUP	TYPE 1		TYPE 2			TYPE 3			TYPE 4	
	FIRE PROOF		NON-COMBUSTIBLE			EXTERIOR MASONRY WALLS			FRAME	
			PROTECTED		UNPROTECTED	H T MILL	ORDINARY JOISTED		PROTECTED	UNPROTECTED
	(Note b)						PROTECTED	UNPROTECTED		
(Note a)	1A	1B	2A	2B	2C	3A	3B	3C	4A	4B
A1A Assembly with stage		6s.23m $1340m^2$	4s.15m $1060m^2$	2s.9m $700m^2$	1s.6m $446m^2$	2s.9m $670m^2$	2s.9m $610m^2$	1s.6m $446m^2$	1s.6m $474m^2$	NP
A1B Theatre without stage			5s.20m $1850m^2$	3s.12m $1220m^2$	2s.9m $780m^2$	3s.12m $1170m^2$	3s.12m $1070m^2$	2s.9m $780m^2$	1s.6m $830m^2$	1s.6m $390m^2$
A2 Assembly, night clubs etc.		4s.15m $670m^2$	3s.12m $530m^2$	2s.9m $350m^2$	1s.6m $223m^2$	2s.9m $335m^2$	2s.9m $305m^2$	1s.6m $223m^2$	1s.6m $246m^2$	1s.6m $110m^2$
A3 Assembly Lec. Halls – Res., Term. rec. centres			5s.20m $1850m^2$	3s.12m $1220m^2$	2s.9m $780m^2$	3s.12m $1170m^2$	3s.12m $1070m^2$	2s.9m $780m^2$	1s.6m $830m^2$	1s.6m $390m^2$
A4 Ass. Churches, Schools. (Note c)			5s.2m $3180m^2$	3s.20m $2090m^2$	2s.9m $1340m^2$ (Note d)	3s.12m $2008m^2$	3s.12m $1840m^2$	2s.9m $1340m^2$ (Note d)	1s.6m $1420m^2$ (Note d)	1s.6m $670m^2$
B Business			7s.26m $3180m^2$	5s.20m $2090m^2$	3s.12m $1340m^2$	5s.20m $20C8m^2$	4s.15m $1840m^2$	3s.12m $1340m^2$	3s.12m $1420m^2$	2s.9m $670m^2$

TABLE 3.206 (cont'd)

TYPE OF CONSTRUCTION

OCCUPANCY GROUP	TYPE 1		TYPE 2			TYPE 3			TYPE 4	
	FIRE PROOF		NON-COMBUSTIBLE			EXTERIOR MASONRY WALLS			FRAME	
	(Note b)		PROTECTED		UNPROTECTED	H T MILL	ORDINARY JOISTED PROTECTED	ORDINARY JOISTED UNPROTECTED	PROTECTED	UNPROTECTED
(Note a)	1A	1B	2A	2B	2C	3A	3B	3C	4A	4B
F Factory, Industrial			6s.23m 2120m^2	4s.15m 1400m^2	2s.9m 900m^2	4s.15m 1340m^2	3s.12m 1227m^2	2s.9m 890m^2	2s.9m 948m^2	1s.6m 445m^2
H High Hazard (Note e)	5s.20m 1560m^2	3s.12m 1340m^2	3s.12m 1060m^2	2s.9m 700m^2	1s.6m 446m^2	2s.9m 670m^2	2s.9m 610m^2	1s.6m 446m^2	1s.6m 474m^2	NP
I-1 Institutional, Restrained		6s.23m 1670m^2	4s.15m 1324m^2	2s.9m 870m^2	1s.6m 558m^2	2s.9m 834m^2	2s.9m 766m^2	1s.6m 558m^2	1s.6m 585m^2	NP
I-2 Institutional Incapacitated		8s.28m 2008m^2	4s.15m 1590m^2	2s.9m 1045m^2	1s.6m 670m^2	2s.9m 1000m^2	2s.9m 900m^2	1s.6m 670m^2	1s.6m 710m^2	NP
M Mercantile			6s.23m 2120m^2	4s.15m 1400m^2	2s.9m 900m^2	4s.15m 1340m^2	3s.12m 1227m^2	2s.9m 890m^2	2s.9m 948m^2	1s.6m 445m^2
R-1 Residential Hotels			9s.30m 2120m^2	4s.15m 1400m^2	3s.12m 900m^2	4s.15m 1340m^2	4s.15m 1227m^2	3s.12m 890m^2	3s.12m 948m^2	2 1/2s.10m 445m^2
R-2 Residential multi-family			9s.30m 2120m^2	4s.15m 1400m^2 (Note f)	3s.12m 900m^2	4s.15m 1340m^2 (note f)	4s.15m 1227m^2	3s.12m 890m^2	3s.12m 948m^2	21/2s.10m 445m^2

TABLE 3.206 (cont'd)

TYPE OF CONSTRUCTION

OCCUPANCY GROUP	TYPE 1 FIRE PROOF (Note b)		TYPE 2 NON-COMBUSTIBLE			TYPE 3 EXTERIOR MASONRY WALLS			TYPE 4 FRAME	
			PROTECTED		UNPROTECTED	H T MILL	ORDINARY JOISTED PROTECTED	ORDINARY JOISTED UNPROTECTED	PROTECTED	UNPROTECTED
(Note a)	1A	1B	2A	2B	2C	3A	3B	3C	4A	4B
R-3 Residential 1 and 2 family			4s.15m $2120m^2$	4s.15m $1400m^2$	3s.12m $900m^2$	4s.15m $1340m^2$	4s.15m $1227m^2$	3s.12m $890m^2$	3s.12m $948m^2$	21/2s.10m $445m^2$
S-1 Storage: moderate (Notes gand h)			5s.20m $1850m^2$	4s.15m $1220m^2$	2s.9m $78m^2$	4s.15m $1170m^2$	3s.12m $1070m^2$	2s.9m $780m^2$	2s.9m $830m^2$	1s.6m $390m^2$
S-2 Storage-low			7s.26m $3180m^2$	5s.20m $2090m^2$	3s.12m $1340m^2$	5s.20m $2008m^2$	4s.15m $1840m^2$	3s.12m $1340m^2$	3s.12m $1420m^2$	2s.9m $670m^2$

TABLE 3.207
PER CENT REDUCTION OF AREA LIMITS

NO. OF STORIES	TYPE OF CONSTRUCTION								
	1A & 1B	2A	2B	2C	3A	3B	3C	4A	4B
1	None	None			None				
2	None	None			None				
3	None	5%			20%				
4	None	10%			20%				
5	None	15%			30%				
6	None	20%			40%				
7	None	25%			50%				
8	None	30%			60%				
9	None	35%			70%				
10	None	40%			80%				

3.208 Unlimited Areas

3.208.1 ONE-STOREY BUILDINGS - In other than frame construction, the area of all buildings of assembly (occupancy group A-3), business (B), factory and industrial (F), mercantile (M) and storage (S) occupancy groups not including high hazard uses, which do not exceed one (1) storey or 26 metres in height shall not be limited; provided the exitway facilities comply with the provisions of Part 3 Section 5 of this code, and an automatic fire suppression system is provided complying the the provisions of Part 3 sub-section 3.703 and the building is isolated as specified in clause 3.208.4, except of Type 2 or Type 3-A construction used exclusively for storage of non-combustible material, not packed or crated in combustible material.

3.208.2 SCHOOL BUILDINGS - One (1) storey school buildings of type 2, 3-A and 3-B construction may be unlimited in area when a direct exitway to the outside of the building is provided from each classroom and the building is equipped with an approved automatic fire suppression system throughout. A fire separation shall be provided on all sides of such buildings as specified in clause 3.208.4 (and table 3.208 below).

3.208.3 INDOOR RECREATION BUILDINGS - Indoor participant sport areas such as tennis courts, skating rinks, swimming pools and other sports clubs may be unlimited in area and exempt from the automatic fire suppression system requirements, providing:

(a) direct exitways to the outside are provided for all the occupants of the recreation area;

(b) the recreation area is conspicuously posted as to use and occupancy load;

(c) the building is equipped with a manual fire alarm system; and

(d) all other areas are equipped with an automatic fire suppression system.

3.208.4 SPECIAL INDUSTRIAL USES - All buildings designed to house low hazard industrial processes, including among others, the production and distribution of electric, gas, or steam power and rolling mills and foundries, requiring large areas and unusual heights to accommodate craneways, or special machinery or équipment, shall be exempt from the height and area limitations of Table 3.206.

3.208.5 FIRE SEPARATION OF BUILDINGS - The minimum fire separation on any side on one (1) storey buildings of unlimited area shall be determined by the type of construction and fire-resistance rating of the exterior wall adjacent thereto as specified in the following Table 3.208.

3.208.6 ROOF VENTS FOR SMOKE AND HEAT - The roof system of one (1) storey buildings of unlimited area when of Type 2 or Type 3 construction shall be provided with smoke and heat vents as suggested in Guide for Smoke and Heat Venting given in NFiPA 204-68.

3.208.7 FIRE ACCESS PANELS - Ground level doors or fire access panels as required and specified in Part 2 Section 2 Fire Access Panels, shall be provided and spaced not more than 46 metres apart in exterior walls adjacent to a required fire system separation of less than 12 metres.

TABLE 3.208

FIRE SEPARATION FOR VARIOUS TYPES OF CONSTRUCTION

Type of construction	Fire-resistance rating of exterior bearing walls	Minimum Fire separation*** in metres	Fire-resistance rating of bearing and non-bearing portions of exterior walls	Minimum Fire separation in metres
2A	2hr.	9 m		
2B	1hr.	12 m	2 hr.*	9 m
2C	0hr.	15 m	3 hr.**	9 m
3A	2hr.	12 m	3 hr.**	9 m
3B	2hr.	12 m	3 hr.**	9 m
3C	2hr.	15 m	4 hr.**	9 m

* All exterior wall openings shall be protected with one and one-half hour fire-resistance approved opening protectives.

** All exterior wall openings shall be protected with three hour fire-resistance rated approved opening protectives.

*** When the fire separation exceeds the herein specified minimum, the requirements of Table 3.114 in Part 3 Section 1 of this Code, Row 1 (Exterior walls with fire separation of 10 m or more: bearing) shall apply.

3.209 Height Exceptions (See also under clause 3.208.4 above)

3.209.1 AUTOMATIC FIRE SUPPRESSION SYSTEMS - When a building of other than high hazard (occupancy group H) use is equipped with an approved automatic fire suppression sytems the building may be erected one storey or 6 metres higher than specified in Table 3.206.

3.209.2 AUDITORIA - Auditoriums (occupancy group A-4) of protected or heavy timber (Type 3-A) construction may be erected to 20 metres in height and of unprotected construction to 13.5 metres.

3.210 Street Encroachments

3.210.1 GENERAL - Except as herein provided, a part of any building hereafter erected and additions to an existing building heretofore erected shall not project beyond the boundary lines or beyond the building line when such line is established by the zoning law or any other statute controlling building construction.

3.210.2 BELOW GROUND LEVEL - Any part of a building hereinafter erected, below the ground level, that is necessary for structural support of the building shall not project beyond the boundary lines except that, if so approved by the local controlling authority, the footings of street walls or their supports located at a depth of not less than 2.4 metres below ground level may project not more than 300 millimteres beyond such street boundary line.

3.210.3 ABOVE GROUND LEVEL - All projections hereafter permitted beyond the street boundary line or the building line above ground level shall be so constructed as to be readily removable without endangering the safety of the building.

3.210.4 PROJECTIONS NECESSARY FOR SAFETY - In any specific application, the Building Authority may designate by approved rules or by a dispensation such architectural features and/or accessories which are deemed desirable or necessary for the health or safety of the public and the extent to which they may project beyond the street boundary line or the building line where such is established by statute, subject to all provisions and restrictions that may be otherwise prescribed by law, ordinance or rule of the authorities having jurisdiction over streets or public spaces. (e.g. fire escapes or balconies).

3.210.5 PERMIT REVOCABLE - Any permit granted or permission expressed or implied in the provisions of this Code to construct a building so as to project beyond the street boundary line or building line shall be revocable by the jurisdiction at will.

3.210.6 EXISTING ENCROACHMENTS - Parts of existing buildings and structures which already project beyond the street boundary line or building line may be maintained as constructed until their removal is directed by the proper authorities of the jurisdiction.

3.211 Permissable Street Projections or Encroachments: (With Local Authority Approval)

3.211.1 GENERAL - Subject to such provisions as may be otherwise prescribed by law or ordinance, or by rule of the muinicipal authorities having jurisdiction over streets, highways, and public spaces, the following projections, as described in clauses 3.211.2 through 3.211.11, shall be permitted beyond the street lot line or the building line, as the case may be, but as intimated above, the Local Authority has the right to withhold approval for any or all of these.

3.211.2 CORNICES AND EAVES - Main cornices or roof eaves located at least three and one-half metres (3.5 m) above the curb level, shall project not more than one-half of one metre (0.5 m)

3.211.3 ARCHITECTURAL DECORATIONS - Belt courses, lintels, sills, architraves, pediments and similar architectural decorations shall project not more than 100 millimetres when less than 3 metres above the curb level, and not more than 250 millimetres when 3 metres or more above the curb level.

3.211.4 ORNAMENTAL COLUMNS - Ornamental columns, or pilasters, including the bases and moldings which emphasize the main entrance of the building shall project not more than 250 millimetres.

3.211.5 ENTRANCE STEPS - Entrance steps and doors shall project not more than 300 millimetres and shall be guarded by check pieces not less than one metre high, or shall be located between ornamental columns or pilasters.

3.211.6 ORIEL WINDOWS - Oriel windows with the lowest portion at least 3 metres above the curb level shall project not more than 0.75 m.

3.211.7 BALCONIES - Balconies located at least 3 metres above the curb level shall project not more than one metre, except that when the balcony is required in connection with a fire escape or exterior stairway as an element of a means of egress, the projection may be increased, but in no case shall it exceed 1.25 metres.

3.211.8 ORNAMENTAL LIGHTS, NEON SIGNS, COATS OF ARMS/INSIGNIAS - Whether incorporated in the original building or whether added at some later date any ornamental lights, or neon signs, any painted signs, or any hanging coats of arms or similar may be attached to a building or a verandah/canopy over a footpath (see below under 3.214) provided that the lowest part of such is at least two and one-quarter metres (2.25 m) above footpath or curb level and that the projection from face of building is either

(a) on buildings with no verandahs or canopies, one metre or

(b) on buildings with a verandah or canopy, projection shall be 250 millimetres less than the width of such canopy or verandah.

3.211.9 AWNINGS - Retractable or fixed awnings shall have clearances above the footpath or curb, and shall be installed in accordance with the requirements of sub-section 3.214.

3.211.10 AWNING COVERS OR BOXES - Awning covers or boxes located at leas 2.4 metres above the curb level shall project not more than one metre.

3.211.11 VAULTS, ACCESS GRILLES OR DOORS TO SAME, AND BELOW GROUND ARCHWAYS - In such buildings as hotels, basement restaurants and banks that have below ground level areas projecting out under footpaths, these shall not extend out further than one and one-quarter metres (1.25 m) from the curb or channel line and the construction and use of such areas in the basement of any buildings shall be subject to the terms and conditions laid down by the local controlling authority. In addition, they shall not project beyond the building or boundary line of any building more than 1.25 metres and shall be covered over at the footpath level by an approved grating of metal or other non-combustible material. Where such underground areas or vaults form light wells to larger basement areas under any building often referred to as pavement lights the covers at footpath level shall be of concrete or metal with prismatic safety glass insets and the cover shall be set permanently in place and be completely watertight.

3.212 Permissable Yard and/or Court Encroachments

3.212.1 GENERAL - No part of any building or structure shall extend into side courts, inner courts or yards required for light and ventilation of habitable and occupiable rooms by the provisions of Part 3 Section 4 of this Code, or of the zoning law or other statutes controlling building construction, except as hereinafter provided; but the encroachment shall not exceed twenty (20) percent of the legal area of yard or court, required for light and ventilation purposes.

3.212.2 ROOF EAVES - Roof eaves shall project not more than one metre beyond the face of the wall.

3.212.3 STEPS AND ARCHITECTURAL FEATURES - Steps, window sills, belt courses and similar architectural features, stormwater/downpipes and chimneys shall project not more than one-half of one metre (0.5 m) beyond the face of any wall.

3.212.4 EXTERIOR STAIRWAYS AND FIRE ESCAPES - Outside stairways, smokeproof tower balconies, fire escapes or other required elements of a means of egress shall not project more than one and one-quarter metres (1.25 m) beyond the face of any wall.

3.212.5 SPECIAL OR TEMPORARY PROJECTIONS - The permissable projection beyond street boundary lines shall apply in general to building projections into alleyways, except as may be modified by the Local Authority having jurisdiction or by special dispensation.

3.213 Verandahs/Awnings and Canopies

3.213.1 PERMIT - A permit shall be obtained from the Building Authority for the erection, repair or replacement of any fixed awning, canopy or hood except as provided in clause 3.213.2, and for any retractable awning located at the first storey level and extending over public street or over any portion of a court or yard beside a building serving as a passage from a required exitway or exitway discharge to a public street.

3.213.2 SPECIAL PERMITS - When authorized by a special permit, vestibules and storm doors may be erected for periods of time not exceeding six (6) months in any one (1) year, and shall project not more than one metre (1.0 m) nor more than one-quarter (1/4) the width of the footpath beyond the street boundary line. Temporary entrance awnings may be erected with a minimum clearance of two and one-tenth metres (2.1 m) to the lowest portion of the hood or awning when supported on removable steel or other approved non-combustible supports.

3.213.3 EXEMPTION FROM PERMIT - A permit shall not be required for the erection, repair or replacement of fixed or retractable awnings installed on one and two-family dwellings, unless they project over public property, or for retractable awnings installed above the first storey or where the awning does not project over the public street or over any court or yard serving as a passage from a required exitway to a public street.

3.213.4 RETRACTABLE AWNINGS - There shall be a minimum clearance of two and two-tenths metres (2.2 m) from the footpath to the lowest part of the framework or any fixed portion of any retractable awning, except that the bottom of the valance of canvas may extend down not more than 150 millimetres below this. Retractable awnings shall be securely fastened to the building and shall not extend closer than 300 millimetres from the curb line. They shall be equipped with a mechanism or device for raising and securely holding the awning in a retracted or closed position against the face of the building.

3.213.5 FIXED OR PERMANENT AWNINGS - The clearance from the footpath to the lowest part of any fixed or permanent awning shall be the same as required in clause 3.213.4 above for retractable awnings. fixed or permanent awnings installed above the first storey shall not project more than one and one-quarter metres (1.25 m).

3.213.6 canopies or street verandahs - canopies or verandahs on any building extending over any footpath on a public street or throughfare shall be constructed either of:

(a) a metal framework with an approved waterproof covering cantilevered out from the building with or without metal ties or braces back to a higher level on the building or

(b) a cantilevered reinforced concrete hood securely tied into the building reinforcement and/or steel frame. such verandahs (or canopies) shall not have any supporting columns at the outer end down to street or footpath, nor shall they have any stormwater downpipes leading from outer edge down to the street channel; all such pipes for disposal of rainwater shall be carried down in or on the wall of the building. The horizontal portion of the framework or soffit of reinforced concrete canopy shall not be less than 2.4 metres above the footpath level. the width of canopies/verandahs shall not exceed two and one-half metres (2.5 m) but in no case shall they be wider than 300 millimetres less than the distance from face of building to the curb or water channel at edge of footpath.

3.213.7 SPECIAL APPLICATIONS OF CANOPY/AWNINGS (CAR PORTS AND SIMILAR) - Rigid awnings or canopies not on a street frontage may be erected supporting members down to the gound and used for patios, car ports, summer houses or other similar uses, but they shall comply with the requirements of 3.213.8 below for design and structure. Such structures shall be braced to provide rigidity.

3.213.8 DESIGN AND CONSTRUCTION - Fixed awnings, canopies and similar structures shall be designed and constructed to withstand wind or other lateral loads and live loads as required by Part 2 Section 1 of this Code with due allowance for shape, open construction and similar features that relieve the pressures or loads. Structural members shall be protected to prevent deterioration by rust or other corrosion.

3.213.9 FASCIA WIDTHS - Fascias of all projecting verandahs/ canopies out over footpaths or public streets shall as far as possible, be kept to a uniform width and height above pavement/footpath level. (See under 3.213.6 (b) above). The maximum width of fascias shall be 380 millimetres and the absolute minimum height of bottom edge of fascia above footpath level shall be 2.4 m.

3.213.10 RECOMMENDATION FOR LOCAL BODIES ISSUING PERMITS FOR VERANDAH/HOODS CANOPIES - Where an existing building has a cantilevered, ground-support-free verandah over a footpath, and an application for a permit is being considered for a similar type of verandah on a new adjoining building, it is recommended that the new building owner be required to line up the new hood and fascia and make it of the same width as that on the existing building to give uniformity.

3.213.11 ROOF LIGHTS IN CANOPIES/VERANDAHS - Fixed roof lights or approved skylights are permissible in roofs of shop/street verandahs, but they must be glazed in not less than 6 millimetre thick wired glass or an approved non-combustible clear plastic.

3.214 Temporary Structures

3.214.1 GENERAL - Pursuant to any variance granted by the Board of Appeals under the provisions of Part 1 Section of this Code, the Building Authority may issue a permit for temporary construction as approved by the Board of Appeals. Such permits shall be limited as to time of service, but such temporary construction shall not be permitted for more than one (1) year.

3.214.2 SPECIAL APPROVAL - All temporary construction shall conform to structural strength, fire safety, means of egress, light, ventilation and sanitary requirements of this code necessary to insure that the health, safety and general welfare of the public is protected.

3.214.3 TERMINATION OF APPROVAL - The Building Authority is authorized to terminate such special approval and to order the demolition of any such construction at its discretion, or as directed by any decision of the Board of Appeals.

3.214.4 BUILDERS' SHEDS/OFFICES/TOOL STORES AND SIMILAR - Refer also to 3.203.5 of this section of the Code in relation to type of construction allowed. Permits may be grantd by the Local Building Authority for builders' sheds and offices and other temporary sheds on a building site or partly or wholly on or above a footpath of any public street for the erection of temporary buildings for use during a building contract. The period for which the permit is granted need not be limited to one year as specified under 3.214.1 above, but shall be governed by the period alowed for the building contract plus one month. The permit will be issued as a part of the building permit for the structure being erected on the site.

3.214.5 Any such permit shall give the Local Authority the power to state what protective measures should be taken to prevent any danger to the public, including safety lighting during the hours of darkness; any required handrails, guard-rails or protection walling. All conditions laid down by the Local Authority must be strictly adhered to by the contractor or persons taking out such permit.

3.215 Physically Handicapped or Aged (Refer Also to Part 3 Section 1 Sub-section 3.125)

3.215.1 APPLICABILITY - The provisions of this section shall apply to all levels and areas used by the general public, employees, persons visiting or on the premises for any reason, and shall apply to all occupancy groups except R-3, R-4 and T. It is particularly applicable to transport terminals generally, but especially so where they may be used by patients in transit.

3.215.2 MODIFICATIONS TO CODE: (SPECIAL CIRCUMSTANCES) - Where it can be demonstrated that one (1) or more of the following provisions is not applicable to the proposed use and occupancy, modifications may be sought under the provisions of Part 1 Section 1 of this Code and a dispensation can be obtained from the Local Authority.

3.215.3 SPECIAL REQUIREMENTS

3.215.4 RESIDENTIAL (R-1) USE - At least (1) bedroom unit for every twenty-five (25) bedroom units or fraction thereof in use group R-1 (residential, hotels) buildings shall be made accessible to physically handicapped persons. The bedroom units allocated for the physically handicapped shall be proportionately distributed throughout all types of units. Access to additional floors without public facilities is not required.

3.215.5 RESIDENTIAL (R-2) USE - At least one (1) dwelling unit for every twenty-five (25) dwelling units or fraction thereof in use group R-2 (residential, multi-family) buildings shall be made accessible to physically handicapped persons. The dwelling units allocated for the physically handicapped shall be proportionately distributed throughout all types of units. Laundry and storage facilities shall be accessible from the barrier free units. Access to additional floors without public facilities is not required.

3.215.6 BUILDING ENTRANCE - At least one (1) primary entrance at each ground floor level of a building shall be accessible from the parking lot or the nearest street by means of a walk uninterrupted by steps or abrupt changes in grade and shall have a width of not less than 1.5 metres and a gradient of not more than one in twenty or a ramp meeting the requirements of Part 3 Section 5 of this Code, except for enclosure. This entrance shall comply with the requirements of this section.

3.215.7 PARKING LOTS AND BUILDING APPROACHES - Any parking lot servicing an entrance described in clause 3.215.6 shall have a number of level parking spaces as set forth in the following Table 3.215 identified by signs as reserved for physically handicapped persons. Each reserved parking space shall be not less than three and two-thirds metres (3.66 m) wide.

TABLE 3.215

ACCESSIBLE PARKING SPACES FOR THE PHYSICALLY HANDICAPPED

Total Parking in Lot	Required Number of Accessible Spaces
up to 50	1
51 - 75	2
76 - 100	3
101 - 200	4
201 - 500	5
above 500	1 percent of total No.

3.215.8 PARKING SPACES - Parking spaces for the physically handicapped shall be located as close as possible to elevators, ramps, walkways, and entrances. Parking spaces should be located so that the physically handicapped persons are not compelled to wheel or walk behind parked cars to reach entrances, ramps, walkways and elevators.

3.215.9 CURBS - Where a curb exists between a parking lot surface and a sidewalk surface, an inclined curb approach or a curb cut with a ramp of not steeper gradient than 1 in 12 and width of not less that 1.25 metres shall be provided for wheelchair access.

3.215.10 INTERIOR ACCESS - Interior means of access to all floor levels required to be accessible for the physically handicapped shall be provided by ramps meeting the requirements of Part 3 Section 5 of this Code or elevators, and access to all points on each floor level shall be provided by means of passageways, corridors, and doorways meeting the requirements of the same section.

3.215.11 ELECTRICAL SWITCHES, CONTROLS, FIRE ALARMS AND TELEPHONES - When required by the Local Fire Board or the Local Authority in buildings likely to be used by physically handicapped persons the following provisions are required:

(a) light switches, controls, fire alarms, shall not be located at a greater height above floor level than 1.25 metres -(this is also specified under Section 7 Part 3 clause 3.718.7 of this Code for manual fire alarm switches).

(b) where public or pay telephones are installed, one telephone shall have the dial, coin slot and handset mounted not more than 1.25 metres above floor level.

3.215.12 LIFTS/ELEVATORS REQUIREMENTS - When and as required by the Local Authority in buildings likely to be used by the physically handicapped, and if interior access in multi-storey buildings is provided by elevator(s), at least one (1) elevator shall meet the requirements listed below.

(a) The elevator cab shall have a clear area of not less than two square metres (2 m2) with a minimum dimension of 1.4 m.

(b) The elevator door shall have a minimum clear opening width of 800 millimetres.

(c) The control buttons shall be located not more than 1.4 metres above the floor in manual controlled lifts (without an attendant).

(d) Braille plates shall be provided adjacent to all cab control button and switches.

(e) Braille plates shall be provided for floor designation on each, floor, 1.5 metres above the floor, on the fixed point at the open side of the elevator door.

3.215.13 ACCESS TO PLUMBING FIXTURES -

3.215.14 TOILET ROOMS - At least one (1) toilet room and one (1) fixture within such room shall be accessible to and usable by, physically handicapped persons. A toilet room shall have a clear space whether access way, lobby or passageway beyond the room door swing of not less than 1.5 metres by 1.5 metres.

3.215.15 WATER CLOSET STALL - The clear width between the face of a water closet stall and a wall at entrance shall be not less than 1.25 metres. A water closet stall shall be not less than 1.25 metres wide, 1.65 metres to 2.0 metres deep, and have an out-swinging door at least 800 millimetres wide or an opening at least 800 millimetres wide. Handrails of not less than 25 millimetres O.D. and not more than 32 mm shall be provided on both sides of the water closet that are not less than 1070 millimetres long and mounted 850 millimetres above and parallel to the floor, with the front end positioned 610 millimetres in front of the water closet.

3.215.16 WATER CLOSET - A water closet shall have a seat not higher than 500 mm from the floor, and have a narrow understructure that recedes sharply from the front. The trap shall not extend in front of, or be flush with, the lip of the bowl. Where only one (1) water closet is required in the facility, a standard height model may be used.

3.215.17 URINAL - If toilet rooms for men have a wall mounted urinal the opening of the basin should be 480 millimetres from the floor, or should have floor-mounted urinals that are level with the main floor of the toilet room.

3.215.18 MISCELLANEOUS - A shelf disposal unit, or the lower edge of a mirror shall not be more than 1 metre (1.0 m) above the floor. A towel or other dispenser, or electrical hand dryer shall not be more than 1.25 metres above the floor.

3.215.19 RECOMMENDATIONS - Although not firm requirements for this Code, the following list is for the information of designers and others and the items have been taken from British, Canadian, New Zealand and other codes dealing with the use of buildings, by physically handicapped, aged, or otherwise disabled persons:

(a) Wheelchair confined males are often accompanied by a mother, sister, or daughter, and females confined to wheelchairs are sometimes accompanied by a husband or son - hence toilet and/or wash rooms for the disabled should be treated as "unisex" ones and accessible from passages or lobbies rather than from either male or female toilet areas.

(b) Doors to wash/toilet areas for the handicapped should always open OUT and be equipped with self-closing hinges, rising butts, or door closers. They should have either horizontal or vertical pull/push bars rather than handles, and a suitable indicator catch showing "vacant" or "occupied".

(c) Vertical grip rails as well as horizontal are also recommended.

(d) In toilets and washrooms used frequently by wheelchair patients (i.e. in institutions, rest homes, hospitals) the entrance doors should have a bell or buzzer outside, operated from pull cord or push/button inside for patients to summon aid.

(e) The water closet flushing handle or button as well as toilet paper holder should be placed where they can be reached by a person seated on the wheelchair.

(f) Passages and corridors in buildings used by handicapped should never be less than 1.25 metres wide.

(g) Drinking fountains should not be fully recessed in walls or alcoves and be set lower for wheelchair confined persons.

(h) Floors generally, and especially ramps should have a non-slip covering; and ramps should have handrails on one side.

(i) Handrails should be easy to grip; if circular, a diameter of not less than 45 millimetres, or more than 51 millimetres is preferred.

(j) Viewing platforms or areas in assembly seating accommodations at airport terminals, sport complexes, and the like should have special areas set aside for wheelchair viewers devoid of any other fixed seating arrangements and the areas shall be in such a position as not to interfere with egress ways from seated areas. Access must be by ramps not steeper than 1 in 12.

3.215.20 TURNSTILES AND CHECKOUT LANES - Buildings which utilize turnstiles to control traffic shall provide a clearly marked alternative route for the physically handicapped which is at least 1 metre wide. Similarly buildings with check-out lanes shall provide at least one check-out lane which is one metre in width and clearly marked "for wheelchairs only".

3.215.21 DWELLING UNITS - In the design of dwelling units for the use of physically handicapped persons, the requirements of sub-section 3.214. Clauses 3.214.1 to 3.214.19 shall be generally adhered to. Passageways in dwellings may be 1 metre wide in lieu of 1.25 m, self closing doors are not required, and lever handles set no higher than 1.1 metres above floor may be used in lieu of pull/push bars on doors.

3.215.22 KITCHENS - A clearance of at least 1370 millimetres shall be provided in front of all cabinets, work surfaces, counter tops and appliances. Also knee space shall be provided under the sink to accommodate persons in wheel chairs.

3.215.23 LAUNDRIES - A clearance of at least 1370 millimetres shall be provided in front of laundry tubs, automatic washers and driers.

3.215.24 BEDROOM (S) - At least one bedroom shall be specially designed to allow free movement of a wheel chair within the bedroom. Clothes closets, wardrobe, floors shall be at the same level as the floor of the room they serve.

3.215.25 SIGNS - (See also under Part 3 Section 1 of this Code, clause 3.125.2).

3.215.26 All buildings erected or altered which are required to comply with this Code shall have the international symbol of accessibility for handicapped persons (a stylized person in wheelchair, reproduced below) prominently displayed in positions as follows:

a) At main entrance to the building. If the access ramp for wheelchairs is not actually at the main entrance, then it must have a directing arrow or notice attached e.g. "access via south door - 50 metres".

(b) Adjacent to the actual ramp access if this is not at the main entrance.

(c) On doors of all toilet areas, water closet cubicles, or rest rooms designed for the use of handicapped persons.

(d) Adjacent to lifts/elevators installed for wheelchair access.

(e) On the signs in parking lots reserved for physically handicapped persons - see 3.215.7 above.

NOTE: For reproduction of international sign see page 14 of "Building Standards for the Handicapped" 1975 - Supplement No. 5 to the National Building Code of Canada.

3.216 Special Historic Buildings

3.216.1 APPROVALS - The provisions of this Code relating to the construction, repair, alteration, enlargement, restoration and moving of buildings or structures shall not be mandatory for existing buildings or structures identified and classified by the state and/or local government authority as historic buildings or places of historic interest.

3.216.2 The above exemptions from code requirements are subject to the building controlling authority in the district, zone, or area being satisfied, that, at the time of a building being declared an Historic one, it is reasonably sound and safe in regard to public health, safety, and welfare, and that any repairs, alterations or re-location proposed, comply with regulations as regards fire zones/areas - (see also in this Part 3 Section 2 of Code Clauses 3.202.1 to 3.202.2 inclusive).

CARIBBEAN UNIFORM BUILDING CODE

PART 3
OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 3
SPECIAL USE AND OCCUPANCY REQUIREMENTS
(HIGH HAZARD OCCUPANCIES)

PART 3

SECTION 3

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PART 3 SECTION 3
(in 2 parts: 3 & 3A)

3.300 SPECIAL USE AND OCCUPANCY REQUIREMENTS (HIGH HAZARD OCCUPANCIES)

3.301 Scope

3.301.1 In addition to the general requirements of this Code, specified mainly in Part 3 Sections 1 and 2 which govern the location, construction, and equipment of all buildings and structures, the fire-resistance ratings, the height and area limitations (in tables), the provisions of this Section (3) shall control all buildings designed for high hazard uses and occupancies which involve extreme fire, smoke, toxic gas or explosion risks, and also places of assembly in which people congregate in large numbers, and which are susceptible to panic incidental to crowds.

3.301.2 Chemical plants, packing plants, grain elevators, refineries, flour mills, and other special structures may be constructed in accordance with the recognised practices and requirements of the specific industry.

3.301.3 The Local Building Authority may permit certain such variations from the requirements of this Code, by special dispensation, provided it will result in reasonable and economic construction along with the necessary fire, life, and property safeguards. In granting such dispensations/variations, due regard shall be given to the isolation of the structure and the fire hazard from and to surrounding properties.

3.301.4 This Section 3 is divided into two halves, 3 and 3A, the first half being further divided into sub-sections (a) to (r) dealing with the following aspects.

3.301.5 SPECIAL HIGH HAZARDS - When necessary to resist a higher degree of fire severity than specified herein, for high concentration of combustible contents and for buildings of high hazard uses which exceed five (5) stories or twenty metres (20 m) in height, the Building Authority may require a higher fire resistance rating than the requirements of Table 3.114 (in Part 3 Section 1) governing fire resistance ratings of the various types of construction and protection of structural elements.

3.301.6 OCCUPANCIES INVOLVING EXPLOSION HAZARDS - The following provisions shall apply to all uses involving the storage, manufacture, handling or filling of flammable and volatile solids, liquids, or gases which generate combustible and explosive air vapour mixtures and toxic gases including nitro-cellulose film pyroxylin plastics; grain and other combustible dusts and pulverized fuels; combustible fibres; pyroxylin lacquer spraying operations; liquefied petroleum gases; alcohol, ether and gasoline; flammable dusts and residues resulting from fabrication, grinding and buffing operations, and all other explosion hazard risks.

3.301.7 MEANS OF EGRESS - The means of egress for buildings of hazardous uses and occupancies shall conform to the requirements of Part 3 Section 5 of this Code, except as may be modified by more restrictive provisions of this section for specific uses.

3.301.8 HEATING AND VENTING - The requirements herein prescribed for the installation of heating and venting appliances and equipment for high hazard uses and occupancies shall be construed as supplemental to the provisions of Part 3 Section 4 and Part 4 Section 1 of this Code (Light, Ventilation and Sound Transmission, and Chimneys, Flues and Vent Pipes) - and also any requirements contained in the Mechanical Code (all of Part 4).

3.301.9 EQUIPMENT ROOMS - Heating and ventilating equipment in occupancies involving fire hazards from flammable vapours, dust, combustible fibres or other highly combustible substances shall be installed and protected against fire and explosion hazards in accordance with the Mechanical Code. Rooms containing such equipment shall be segregated by construction of not less than two (2) hour fire-resistance rating except as may be required for specific uses without openings in the enclosure walls and with means of direct ingress and egress from the exterior, or such equipment shall be located in accessory structures segregated from the main building.

3.301.10 FIRE FIGHTING AND EXTINGUISHING EQUIPMENT - All buildings designed for specific hazardous uses shall be protected with approved automatic fire suppression systems or such other fire-extinguishing and auxiliary equipment as herein provided and in accordance with the requirements of Part 3 Section 7 of this Code (Fire Protection Systems).

3.301.11 RESTRICTED LOCATIONS - Except as otherwise specifically approved, high hazard uses shall not be located within the fire limits nor in a building of unprotected frame (Type 4-B) construction, nor in any case within sixty metres (60 m) of the nearest wall of a building classified in a public assembly or institutional occupancy group.

3.302 Explosion Hazards

3.302.1 EXPLOSION RELIEF - Every structure, room or space occupied for uses involving explosion hazards shall be equipped and vented with explosion relief systems and devices arranged for automatic release under predetermined increase in pressure as herein provided for specific uses or in accordance with accepted engineering standards and practice.

3.302.2 VENTING DEVICES - Venting devices to relieve the pressure resulting from explosive air-vapour mixtures shall consist of windows, skylights, vent flues or releasing roof or wall panels which discharge directly to the open air or to a public place or other unoccupied space not less than six metres (6 m) in width on the same lot or section of land. Such releasing devices shall be so located, that the discharge end shall not be less than three metres (3.0 m) vertically and six metres (6.0 m) horizontally from window openings or means of egress facilities in the same or adjoining buildings. The exhaust shall always be in the direction of least exposure and never into the exterior of the building.

3.302.3 AREA OF VENTS - The aggregate clear vent relief area shall be regulated by the type of construction of the building and shall be not less than prescribed below:

(1) Heavy reinforced concrete frame, one square metre (1.0 m^2) to twenty-five cubic metres (25.0 m^3) of volume.

(2) Light structural steel frame and/or ordinary construction, one square metre (1.0 m^2) to twenty cubic metres (20.0 m^3) of volume.

(3) Light wood frame construction, one square metre ($1.0\ m^2$) to sixteen cubic metres ($16.0\ m^3$) of volume.

The combined area of open windows, pivoted sashes or wall panels arranged to open under internal pressure shall not be less than ten (10) percent of the area of the enclosure walls, with not less than fifty (50) percent of the opening arranged for automatic release.

3.302.4 CONSTRUCTION OF RELIEF VENTS - All explosion relief devices shall be of an approved type, constructed of light weight, non-combustible and corrosion-resistive materials, and the discharge end shall be protected with approved screens of not more than nineteen millimetre (19 mm) mesh, arranged to blow out under relatively low pressures.

3.303 Volatile Flammables

3.303.1 STORAGE (INSIDE FOR PROCESSING) - Unless otherwise approved by the Fire Authority, inside storage in process rooms shall be limited to one (1) day's supply in approved sealed containers of not more than twenty-five litre (25 l) capacity or in approved steel barrels or drums of not more than two hundred and fifty litres (250 l) capacity.

3.303.2 HANDLING - Discharge, decanting, or filling operations shall be by pump through an approved system of securely attached and continuous piping or hose lines. In processes requiring the use of open vats or mixing tanks, an approved mechanical ventilating system shall be provided to remove the vapours or to produce a vapour mixture of not more than one (1) percent concentration.

3.303.3 CONSTRUCTION OF ENCLOSURES - Process rooms shall be separated from other uses and occupancies by walls, floors and ceilings of not less than two (2) hours fire-resistance rating with one and one-half (1 1/2) hour fire doors or the approved labelled equivalent complying with Part 3 Section 6 of this Code. The access doors openings to such enclosure(s) shall be provided with non-combustible sills not less than one hundred and fifty millimetres (150 mm) high and the room shall be vented as required in clause 3.303.2 above. Floors shall be waterproofed and drained. There shall be at least two remote paths of egress from the environs of any such enclosure leading to outside of the building (see also Means of Egress - Part 3 Section 5 of this Code) and approved "no smoking" signs shall be exhibited giving a clear space around such enclosure of at least six metres (6 m).

3.303.4 FIRE PROTECTION - First aid fire appliances and/or automatic fire-extinguishing systems shall be provided in accordance with Part 3 Section 7 of this Code. Only electric incandescent lighting shall be used in, and within six metres (6.0 m) of such enclosures and all light fittings shall be of approved gas-tight/explosion-proof type. Provision shall be made to prevent leaking flammable vapours from being exposed to open flames, fires or sparks.

3.303.5 MAIN STORAGE - Main storage systems of volatile flammable liquids shall be constructed and installed in accordance with the U.S. National Fire Protection Association (NFiPA) Codes 30-76, 58-76 and 59-76 or equivalent British Standards until such time as the Caribbean has adopted an acceptable equivalent. Such storage may be:

(i) inside underground
(ii) outside underground
(iii) outside above ground or
(iv) in a separate outside storage building

No outside above ground bulk storage tank shall be located less than ninety metres (90.0 m) from any building of assembly (occupancy group A) or institutional occupancy (group I) building.

NOTE: Certain types of school laboratory e.g. Agricultural or Science laboratories in post-primary or tertiary schools, or special experimental laboratories may be exempted from the above regulations provided that both the Fire Department and the Inspector of Explosives (or his equivalent) have approved of safety measures to be adopted.

3.303.6 OUTSIDE, UNDERGROUND STORAGE TANKS - Outside storage tanks shall be buried underground below the basement level of adjacent buildings, with the top of the tanks not less than 60 centimetres (0.6 m) below ground level, or with a reinforced concrete or other approved structural cover not less than 100 millimetres (100 mm) thick and a 300 millimetre (300 mm) earth cover. The maximum capacity of such tanks shall be limited by their location in respect to adjacent buildings which are not an essential part of the installation and adjacent boundaries as provided in Table 3.303.

3.303.7 When within three metres (3.0 m) of any building not an essential part of the installation, and the top of the tank is above the lowest floor of the building, the capacity of the tank shall be not more than two thousand and five hundred litres (2,500 l). The capacity of storage of combustible liquids other than volatile flammable, as defined herein shall be restricted to five (5) times the values/volumes specified in Table 3.303 following .

TABLE 3.303

CAPACITY OF OUTSIDE UNDERGROUND STORAGE TANKS FOR VOLATILE FLAMMABLE LIQUIDS

Quantity of Storage in Litres	Fire Separation in Metres
13,500	3.0
27,000	6.0
54,000	7.5
90,000	9.0
225,000	12.0
Unlimited	15.0

3.303.8 VENTING OF TANKS - All storage tanks whether inside or outside, above ground or underground, shall be vented to the open air. Each tank shall have a vent pipe independent of all other piping, and of ample size to prevent abnormal pressure build-up during fillings, but not of less internal diameter than thirty-two millimetres (32 mm). Vent pipes shall drain back to the tank. They shall not be closer to any opening in any building than one metre (1.0 m); they shall terminate in a U-bend facing downwards to prevent rain penetrating, and shall be fitted with a flame arrestor.

3.303.9 ANCHORING DOWN OF STORAGE TANKS - In any areas where the ground water level is likely to be above the bottom of the tank at any time of the year, storage tanks are to be securely anchored down in a manner approved by the Local Authority, the Inspector of Explosives, and/or the Local Fire Board. It is preferable that in such areas of high water tables that underground tanks, as well as above ground tanks, be set on saddles, shaped to the contour of the bottom of the tanks, constructed of concrete or brick.

3.303.10 SPECIAL CASES (CENTRAL CITY OR HIGH HAZARD AREAS) - Where the Local body, Fire Authority or Explosives Inspectorate consider it warranted; especially in central city fire areas or other high-hazard areas, underground tanks shall be placed inside concrete pits, on saddles, anchored down, and the whole of the space between the concrete walls of tank pit and the metal tank itself shall be filled to the underside of top slab with dry sand. Such tank shall be constructed preferably in situ, of reinforced concrete not less than one hundred millimetres (100 mm) thick walls and floor and with the top of removable slabs also of reinforced concrete and also one hundred millimetres (100 mm) in thickness.

3.303.11 FILLING OF STORAGE TANKS - All storage tanks, whether above or below ground level shall be filled only through proper fill pipes, terminating outside of buildings at a point at least one metre (1.0 m) from any building opening at the same or any lower level. All fill terminals shall close tight when not in use.

NOTE: Gas or bowser or similar filling stations with roofs supported on steel or concrete columns but with no side walls are exempt from these requirements for filling points.

3.303.12 OUT OF SERVICE UNDERGROUND TANKS - These shall be treated as follows:

(a) Tanks out of service for up to three months - no special precautions, except they must be inspected by an Explosives Inspector or similar responsible officer before they are refilled.

(b) Underground tanks out of service for any longer period than three months but required for use at some later date shall have both the vents and fill terminals capped and sealed with concrete. They too shall be inspected before re-use as required under 3.303.12 (a) above, and

(c) Underground tanks permanently abandoned shall either be completely removed, or be filled with sand. The responsibility for this shall be with the property owner.

3.303.13 INSIDE UNDERGROUND STORAGE - Inside U-ground tanks shall be located not less than six-tenths of one metre (0.6 m) below the level of the lowest floor of the building in which located, or any other building within a radius of three metres (3.0 m) of the tank. Such tanks shall not be located under the footpath or beyond the building line. It shall be unlawful to cover any tanks from sight until after inspection and test and written approval of the Building Authority/Explosives Inspector. The maximum limit of individual tank capacity shall be not more than two thousand five hundred litres (2,500 l) and the entire system shall be subject to special approval of the Building and Fire Officials.

3.303.14 OUTSIDE ABOVE GROUND STORAGE - Above ground tanks shall be located only outside the fire limits and the capacity, location, construction and exposures shall be in accordance with applicable standards or as approved by the Local Authority, Explosives Inspector and Local Fire Authority.

3.303.15 OUTSIDE STORAGE HOUSE - All outside storage houses shall be constructed of non-combustible (Type 2) construction or better. Openings shall not be permitted in the enclosure walls within three metres (3.0 m) of adjoining property lines or with a fire exposure of less than three metres (3.0 m) from any building or structure not part of the installation.

3.303.16 SPECIAL RESTRICTIONS - The Building or Fire Authority may require greater fire separations or may limit storage capacities under severe exposure hazard conditions when necessary for public safety.

3.304 Fire Prevention Code

3.304.1 INSPECTIONS - All buildings involving the use and handling of flammable or explosive materials, places of assembly and other hazardous uses and occupancies shall be inspected in accordance with the Fire Prevention Code, the requirements of which are covered in this Section 3 and also by Part 3 Sections 5, 6 and 7 of this Code. Such inspection shall be made to insure compliance with the provisions of the Fire Prevention Code in respect to protection against fire and panic; maintenance of exitways and operation of fire door assemblies; fire protection systems; standpipes; hydrant and fire suppression systems; fire-alarm signalling and central station alarm systems; conduct of fire drills and fire brigades; and all special fire-extinguishing equipment.

3.304.2 HOUSEKEEPING - Periodic inspections of existing uses and occupancies shall be made to insure maintenance of good housekeeping conditions including the removal of waste and rubbish safe arrangement and storage of merchandise and other contents; proper segregation of hazardous processes; handling of volatile flammables; avoidance of dangerous congestion and maintenance of all means of egress clear of obstructions; and the safe operation of all places of public assembly in which combustible scenery and hazardous equipment are in use while open to the public.

3.304.3 COORDINATION OF INSPECTIONS - The Building, Fire, and Health Officials and other administrative agencies of the jurisdiction to whom the authority is delegated to inspect buildings and structures in respect to the maintenance of safe conditions of use and occupancy shall immediately notify the respective official of any violation of the provisions of this Code or the Fire Prevention and Health Rules and Regulations.

3.305 Special Permits and Certificate of Fitness

3.305.1 PERMITS - Any hazardous or dangerous industry, trade, occupation or use which involves the transportation, storage or handling of explosive, flammable, combustible, or other substance involving fire or life hazards shall not be conducted without a permit from the fire authority and Explosives Inspector prescribing the conditions and requirements necessary to secure the public safety.

3.305.2 CERTIFICATE OF FITNESS - Before any equipment involving fire or life hazard is placed in operation, the supervisor or operator shall secure a certificate of fitness from the administrative official certifying to the qualification of the person to whom such certificate is issued. Certificates of fitness shall be required for the operation of boilers and unfired pressure vessels and for the conduct of all high hazard uses involving the storage, use or handling of flammable volatile liquids, materials and mixtures, liquefied gases and compressed gases under a pressure of more than one hundred kilopascals (100 k Pa) and all acid and liquid chemicals of a combustible and explosive character. All certificates of fitness may be terminated for cause at any time, and shall be renewed at intervals of not more than one (1) year.

3.306 Existing Buildings

3.306.1 SPECIAL PERMIT FOR EXISTING USES - Any existing hazardous use which was heretofore authorized by a permit issued under the provisions of law or the regulations of the Fire Authority may be continued by special permit provided the continuance of such use or occupancy does not endanger the public safety.

3.306.2 EXISTING USE PROHIBITED - An existing building of frame (Type 4) construction which is more than two (2) stories in height or more than four hundred and fifty square metres ($450m^2$) in area; or of non-fireproof (Type 3) construction which is more than four (4) stories in height shall not be continued in use or hereafter occupied for the manufacture of pyroxylin plastics or similar materials of high fire hazard and explosive characteristics.

3.306.3 CHANGE OF USE FOR PLACES OF ASSEMBLY (A OCCUPANCIES) - An existing building or part thereof shall not be altered or converted into a place of assembly unless it complies with all provisions of this Code applicable to places of public assembly hereafter erected and when an existing building heretofore used as a place of public assembly is altered and the cost of such alteration is more than fifty (50) percent of the physical value of the building, such alterations shall comply as nearly as is practicable with the provisions of this Code which govern the arrangement and construction of seats, aisles, passageways, stage and appurtenant rooms, fire-fighting and extinguishing equipment and the adequacy of means of egress.

3.306.4 INCREASE IN OCCUPANCY LOAD - Whenever the occupancy load of an existing place of public assembly is increased beyond the approved capacity of its exitways, the building or part thereof shall be made to comply with the requirements for a new building hereafter erected for such public assembly use/occupancy.

3.306.5 SWIMMING POOLS - CHANGE OF USE - An existing pool used for swimming or bathing or accessory equipment or part thereof shall not be altered or converted for any other use unless it complies with all provisions of this Code applicable to the use intended. Existing swimming pools may be continued without change; provided safety requirements of 3.330.16 and 3.330.17 below are observed as required by the Building Authority.

3.307 Liquefied Petroleum Gases (LPG)

3.307.1 GENERAL - The provisions of this sub-section shall apply to the design, construction, location, installation and operation of propane, butane and other petroleum gas facilities, normally stored in liquid state under pressure for use in all buildings. Refineries, tank farms and utility gas plants shall be subject to special approvals in accordance with accepted engineering practice and/or as covered in any specific standards or codes for the use of.

3.307.2 There are many other types of flammable gases generally stored and used from pressure bottles, such as liquid oxygen, acetone/acetylene, hydrogen, methane, CNG (compressed natural gas) and others with differing "trade names" of a similar character. All of these gases and the handling, storage and usage of them shall be governed/ controlled by this sub-section, 3.307.

3.307.3 CLASSIFICATION OF SYSTEMS - Systems for the storage and use of liquefied petroleum gases shall be classified as: cylinder or bottled gas systems; above ground tank systems other than bottled gas; and underground tank systems.

3.307.4 BOTTLED GAS - A container or cylinder of bottled gas for domestic or commercial use shall not exceed five thousand litre (5,000 l) equivalent water capacity; and such container shall be tested and approved by an accredited testing authority and shall be identified in accordance with Department of Transportation (DOT) regulations or equivalent standards or special requirements. The cylinders shall be installed above ground, with valves, flexible connectors, piping and safety devices in accordance with the approved rules; except that such containers, when approved by the building and fire authority, may be installed for use inside buildings for industrial purposes or in connection with construction, repair, or alteration operations.

3.307.5 ABOVE GROUND TANK SYSTEMS OTHER THAN BOTTLED GAS - All above ground tank systems other than cylinder or bottled gas shall be located with respect to boundary/lot lines and adjacent buildings on the same lot as specified in Table 3.307. The tanks shall be constructed and tested in accordance with the regulations of the mechanical Code for unfired pressure vessels; and the installation, valves, accessories, piping, vapourizers and safety devices shall be in accordance with accepted engineering practice. Bulk storage shall not be permitted within the fire limits.

3.307.6 SPECIAL RESTRICTIONS - The Building Authority, Explosives Inspectorate and/or Fire Authority may require greater fire separations or greater limitations of storage capacity than is specified in Table 3.309, when necessary for public safety.

3.307.7 UNDERGROUND TANK SYSTEMS - Underground tank systems shall be buried at least six-tenths of one metre (0.6 m) below ground level. When required, such tanks shall be anchored or weighted to prevent floating as specified under 3.303.9 and 3.303.10 for volatile liquids storage tanks. All containers shall be given an approved protective coating of hot dip galvanizing red lead and asphalt, or other approved corrosion-resistive protection. The fire separation from boundary lines and other buildings on the same lot shall comply with Table 3.307 below.

TABLE 3.307

SEPARATION DISTANCES FOR TANK CONTAINER SYSTEMS FOR LIQUID PETROLEUM (AND OTHER) GASES

EQUIVALENT WATER CAPACITY PER CONTAINER (IN LITRES)	MINIMUM DISTANCES (IN METRES)		
	CONTAINERS		BETWEEN ABOVE GROUND CONTAINERS
	ABOVE GROUND	BELOW GROUND	
Less than 550 (note A)	3	none (Note B)	none
550 to 1100	3	3	none
1100 to 2200	3	3	1
2200 to 8800	7.5 (Note C)	7.5 (Note C)	1
8800 to 115,000	15	15	2
115,000 to 2,000,000	15	22.5	1/4 of sum of diameters of adjacent containers
above 2,000,000	15	45	

3.307.8 LABELLING - All inlet and outlet connections except safety relief valves, level and pressure gauges shall be labelled to designate whether they communicate with vapour or liquid space and the tanks shall be marked with a securely attached label and nameplate identifying the system, working pressure, vapour pressure of the contents and permissible liquid level in accordance with accepted engineering practice.

3.307.9 INSTRUCTIONS - Complete installation, operation and maintenance procedure instructions shall be supplied by the manufacturer/supplier for the personnel responsible for the use of the system/installation.

3.307.10 GROUNDING (EARTHING) - All above ground tanks exceeding five thousand litres (5,000 l) equivalent water capacity shall be permanently and effectively grounded (electrically).

NOTES APPLICABLE TO TABLE 3.307

Note A: At a consumer site, if the aggregate water capacity of a multi-container installation comprised of individual containers having a water capacity of less than 550 litres is 1100 litres or more, the minimum distance shall comply with the appropriate portion of this table applying the aggregate capacity rather than the capacity per container. If more than one such installation is made each installation shall be separated from any other installation by at least 7.5 metres. Do not apply the MINIMUM DISTANCES BETWEEN ABOVE GROUND CONTAINERS to such installations.

Note B: The following shall apply to above ground containers installed alongside of buildings:

1. Portable DOT cylinder specification containers shall be located and installed so that the discharge from the container safety relief device is at least one metre (1.0 m) horizontally away from any building opening below the level of such discharge, and shall not be beneath any building unless this space is well ventilated to the outside and is not enclosed for more than fifty (50) percent of its perimeter. The discharge from container safety relief devices shall be located not less than two metres (2.0 m) in any direction away from openings into sealed combustion system appliances or mechanical ventilation air intakes.

2. ASME specification containers of less than 550 litres water capacity shall be located and installed so that the discharge from safety relief devices shall not terminate in or beneath any building and shall be located at least two metres (2.0 m) horizontally away from any building opening below the level of such discharge, and not less than two metres (2.0 m) in any direction away from openings into sealed combustion system appliances or mechanical ventilation air intakes.

3. The filling connection and the vent from liquid level gauges on either DOT or ASME containers filled at the point of installation shall be not less than three metres (3.0 m) in any direction away from air openings into sealed combustion system appliances or mechanical ventilation air intakes.

Note C: This distance may be reduced to not less than three metres (3.0 m) for a single container of five thousand litres (5,000 l) water capacity or less provided such container is at least seven and one half (7.5m) metres from any other LP-Gas container of more than five hundred and fifty litres (550 l) water capacity.

3.308 **Pyroxylin Plastics**

3.308.1 GENERAL - The provisions of this sub-section shall regulate all building, structures and parts thereof used for the storage, handling or fabrication of pyroxylin plastic whether in raw material, process, finished product or scrap.

3.308.2 EXCEPTIONS - The provisions of this sub-section shall not apply to the manufacture, use or storage of nitro-cellulose film or the incidental storage of articles manufactured from pyroxylin plastics offered for sale in mercantile buildings (see Part 3 Section 1, 3.109 of this Code).

3.308.3 RESTRICTIONS - A permit for the storage or manufacture of pyroxylin plastics, except as specified in 3.308.2 above shall not be issued for a building or structure hereafter erected, altered or used which is occupied or located as described in the following clauses 3.308.4 through to and including 3.308.8.

3.308.4 PLACE OF ASSEMBLY (OCCUPANCY GROUP(S) A) - Within fifteen metres (15.0 m) of the nearest wall of a school, theatre, or other place of public assembly.

3.308.5 RESIDENTIAL BUILDING - As a residential building, occupancy group R-1, R-2 or R-3.

3.308.6 HIGH HAZARD USES - In quantities exceeding four hundred and fifty kilograms (450 kg) in buildings where paints, varnishes or lacquers are manufactured, stored or kept for sale, or where matches, resin, oils, hemp, cotton, or any explosives are stored or kept for sale.

3.308.7 OTHER FLAMMABLE MATERIALS - Where dry goods, garments or other materials of a highly flammable nature are manufactured in any portion of the building above that used for nitro-cellulose products.

3.308.8 FACTORY BUILDING - In quantities exceeding forty-five kilograms (45 kg) in any tenant factory building (occupancy group F) in which more than five (5) people are employed, or likely to congregate on one floor at any one time.

3.308.9 INSIDE STORAGE - All pyroxylin raw material and products intended for use in further manufacture shall be stored as herein provided in the following clauses 3.308.10 to and including 3.308.15.

3.308.10 CABINETS - Quantities of more than twelve kilograms (12 kg) and not more than two hundred and twenty-five kilograms (225 kg) shall be stored in approved cabinets constructed of non-combustible materials but the total quantity of storage shall not be more than four hundred and fifty kilograms (450 kg) in any workroom or space enclosed in floor, walls and ceilings of not less than two (2) hours fire-resistance rating.

3.308.11 VAULTS - Quantities of more than forty-five kilograms (45 kg) and not more than four thousand five hundred kilograms (4,500 kg) shall be stored in vaults enclosed in floors, walls and ceilings of not less than four (4) hours fire-resistance rating. The interior storage volume of the vault shall be not more than forty cubic metres (40 m^3) and the vault shall be constructed vapour and gas-tight in accordance with the approved rules, with one and one-half (1 1/2) hour vapour-tight fire doors or the approved labelled fire door assembly equivalent on each side of the door opening. The vault shall be drained and provided with scuppers.

3.308.12 TOTE BOXES AND SCRAP CONTAINERS - During manufacture, pyroxylin materials and products not stored in finished stock rooms, cabinets of vaults shall be kept in approved covered non-combustible tote boxes. Scrap and other refuse material shall be collected in approved non-combustible containers in quantities not greater than one hundred and fifty kilograms (150 kg) and removed at frequent intervals as directed by the Fire Officials.

3.308.13 VENTILATION - Each separate compartment in storage vaults shall be vented directly to the outer air through flues complying with the requirements of the applicable Mechanical Code for low temperature chimneys, or exterior metal smokestacks, or as otherwise provided in the approved rules. The vent shall discharge not less than one metre (1.0 m) above the roof of the building or on a street, court or other open space not less than fifteen metres (15 m) distance from any other opening in adjoining walls which are not in the same plane, nor nearer than seven and one-half metres (7.5 m) vertically or horizontally to an exterior stairway, fire escape or exitway discharge. The area of the vent shall be not less than two square centimetres (2 cm^2) for each one kilogram (1.0 kg) of pyroxylin stored.

3.308.14 STRUCTURAL STRENGTH - The floors, walls, roof and doors of all vaults, store-rooms or buildings used for the storage or manufacture of pyroxylin materials and products shall be designed to resist an inside pressure load of not less than fifteen kilopascals (15.0 k Pa).

3.308.15 FIRE PROTECTION - Vaults located within buildings for the storage of pyroxylin shall be protected with an approved automatic sprinkler system (see Part 3 Section 7 of this Code - sub-section 3.705) capable of discharging eighty litres (80 l) per minute per one square metre (1.0 m^2) over the area of the vault.

3.308.16 ISOLATED STORAGE BUILDING - Pyroxylin products in quantities greater than permitted for interior storage shall be housed in isolated storage buildings. Such buildings shall not be used for any purpose other than packing, receiving, shipping and storage of pyroxylin plastics unless otherwise approved by the Building Authority/Fire Authority/Explosives Inspectorate.

3.308.17 CAPACITY - The maximum storage in any fire area enclosed in construction of four (4) hours fire-resistance rating shall be not greater than forty five tonnes (45 t). The storage capacity of the building and its separation from lot lines and other buildings on the same lot shall be limited as provided in Table 3.308. When equipped with an approved automatic sprinkler system complying with the provisions of Part 3 Section 7 of this Code, and as herein modified, the exposure distances may be decreased fifty (50) percent. Such systems shall be designed in accordance with section 2061 of the NFiPA 42 or other applicable standards.

TABLE 3.308
EXPOSURE DISTANCE FOR PYROXYLIN STORAGE BUILDINGS

Maximum Storage (in Kilograms)	Fire Separation from Boundaries or other Buildings (in Metres)
500	12
1,000	15
2,000	20
5,000	30
10,000	35
20,000	50
50,000	70
above 50,000	100

3.308.18 FIRE PROTECTION (i) HEATING EQUIPMENT - All radiators, heating coils, piping and heating apparatus shall be protected with approved non-combustible mesh to maintain A CLEARANCE OF ONE HUNDRED AND FIFTY MILLIMETRES (150 mm) of all pyroxylin products from such equipment. All piping and risers within one metre of the floor shall be insulated with approved non-combustible covering unless protected with wire guards.

3.308.19 FIRE PROTECTION (ii) ELECTRICAL WIRING AND EQUIPMENT - All electrical wiring and equipment shall conform to the provisions of Part 4 Section 2 of this Code, and also to the requirements/provisions of the NFiPA 42 for Pyroxylin Plastics.

3.308.20 FIRE PROTECTION (iii) STANDPIPES FOR - First aid standpipes shall be provided for each four hundred and fifty square metres (450 m^2) of floor area, equipped with thirty-eight millimetre (38 mm) hose, complying with Part 3 Section 7 of this Code.

3.308.21 FIRE PROTECTION (iv) AUTOMATIC SPRINKLERS - All manufacturing and storage spaces and vaults where required shall be protected with an approved automatic sprinkler system as herein specified and with fire pails and portable fire extinguishers complying with Part 3 Section 7 of this Code and the approved rules.

3.308.22 SPECIAL PROTECTION - Special chemical extinguishers and other first-aid fire appliances shall be provided around motors and other electrical equipment in accordance with the approved rules.

3.309 Use and Storage of Flammable Film (Including Requirements for Projection Rooms)

3.309.1 PERMIT REQUIRED - A permit for handling use, storage or recovery of flammable film shall not be issued for any building located as specified in clauses 3.308.3, except that those restrictions shall not apply to the screening and projection rooms of theatres and other places of amusement or instruction. It shall be unlawful to store, stock or use any nitro-cellulose or other flammable film in quantities of more than six hundred metres (600 m) in length or more than five kilograms (5 kg) in weight unless approved by the fire authority. All installations shall comply with the applicable standards and this Code.

3.309.2 STORAGE - Other than motion picture projection and rewind rooms, or as herein specifically exempted all rooms in which flammable film is stored or handled shall be enclosed in not less than two (2) hour fire-resistive construction complying with the provisions of Part 3 Section 6 of this Code (Fire-Resistive Construction Requirements). All film, except when in process or use, shall be kept in approved closed containers.

3.309.3 CABINETS - Flammable film in amounts of twelve to five hundred kilograms (12 to 500 kg) shall be stored in approved non-combustible cabinets constructed and vented in accordance with the approved rules. One (1) cabinet shall not contain more than one hundred and seventy-five (175) kilograms. All cabinets with a capacity of more than thirty-five (35) kilograms shall be equipped with not less than one (1) automatic sprinkler head.

3.309.4 VAULTS - Flammable film in amounts greater than five hundred kilograms shall be kept in vaults constructed as provided in sub-section 3.308 except that the interior storage volume shall not exceed twenty cubic metres 20 m^3.

3.309.5 ROOMS - Unexposed film may be stored in the original approved shipping cases complying with the rules of the relevant local standard of Department of Transportation (DOT) in rooms equipped with an approved automatic sprinkler system complying with the provisions of clauses under 3.308.15 above.

3.309.6 VENTILATION OF STORAGE ROOMS - Storage rooms shall be ventilated as specified in clause 3.308.13 above, with the vents arranged to open automatically in the event of fire in accordance with the approved rules.

3.309.7 HEATING AND/OR COOLING (STORAGE ROOMS) - All heating or cooling (air-conditioning) equipment and installations shall conform to the requirements of 3.308.19 above. The duct systems of warm air heating and air-conditioning systems shall comply with Part 5 Section 2 of this Code, and shall be protected with automatic fire dampers to cut off all rooms in which film is handled from all other rooms and spaces in the building. The heating of film vaults shall be automatically controlled to a maximum temperature of twenty-one degrees Celsius (21 degrees C).

3.309.8 FIRE PROTECTION OF STORAGE AREAS - Approved automatic sprinkler systems shall be provided in all buildings and structures and parts thereof in which flammable film is stored or handled in amounts of more than twenty-four kilograms (24 kg) and herein specifically required, except in projection booths and rewind rooms conforming to the requirements of clause 3.309.9 below. First aid fire-extinguishing and auxiliary fire-fighting equipment shall be provided in accordance with Part 3 Section 7 of this Code and the approved rules adopted thereunder.

3.309.9 PROJECTION ROOMS REQUIRED (SCOPE) - The provisions of this section shall apply to rooms in which ribbon-type cellulose acetate or other safety film is used in conjunction with electric arc, xenon or other light source projection equipment which develops hazardous gases, dust or radiation.

Where cellulose nitrate film is used, projection rooms shall comply with NFiPA 40, and/or other applicable standards. Every motion picture machine projecting film as mentioned within the scope of this sub-section shall be enclosed in a projection room. Appurtenant electrical equipment, such as rheostats, transformers and generators, may be within the projection room or in an adjacent room of equivalent construction. There shall be posted on the outside of each projection room door and within the projection room itself a conspicuous sign with twenty-five millimetre (25 mm) block letters stating: SAFETY FILM ONLY PERMITTED IN THIS ROOM.

3.309.10 CONSTRUCTION OF PROJECTION ROOMS - Every projection room shall be of permanent construction consistent with the construction requirements for the type of building in which the projection room is located. The room shall have a floor area of not less than seven and one-half square metres (7.5 m^2) for a single machine. Each motion picture projector, floodlight, spotlight, or similar piece of equipment shall have a clear working space of not less than three-quarters of a metre (0.75 m) by three-quarters of a metre (0.75 m) at each side and at the rear of it, but only one such space shall be required between two (2) adjacent projectors (or pieces of equipment). The projection room, and any rooms appurtenant thereto shall have a ceiling height of not less than two and one-quarter metres (2.25 m). The aggregate of openings for projection equipment shall not exceed twelve and one-half percent (12 1/2%) of the area of the wall between the projection room and the auditorium. All openings shall be provided with glass or other approved material, so as to completely close the opening.

3.309.11 MEANS OF EGRESS FROM PROJECTION ROOMS - The method of exiting from any projection room shall be as required in Part 3 Section 5 of this Code (Means of Egress).

3.309.12 VENTILATION OF PROJECTION ROOMS AND EQUIPMENT -Ventilation shall be provided in accordance with the provisions of this Section 3.309.13 to 3.309.17.

3.309.13 SUPPLY AIR - Each projection room shall be provided with two (2) or more separate fresh air inlet ducts with screened openings terminating within three hundred millimetres (300 mm) of the floor and located at opposite ends of the room. Such air inlets shall be of sufficient size to permit an air change every three (3) minutes. Fresh air may be supplied from the general building air-conditioning system; but when this is done, it shall be arranged that the projection booth will continue to receive one (1) change of air every three (3) minutes, regardless of the status of the general air-conditioning system.

3.309.14 EXHAUST AIR - Each projection room shall be provided with one (1) or more exhaust air outlets which may be manifolded into a single duct outside the booth. Such outlets shall be so located as to insure circulation throughout the room. Projection room exhaust air systems shall be independent of any other air systems in the building. Exhaust air ducts shall terminate at the exterior of the building in such a location that the exhaust air cannot be readily recirculated into the supply air system The exhaust system shall be mechanically operated and of such a capacity as to provide a minimum of one (1) change of air every three (3) minutes. The blower motor shall be outside the duct system. The projection room ventilation system may also serve appurtenant rooms, such as the generator room and the rewind room.

3.309.15 PROJECTION EQUIPMENT VENTILATION - Each projection machine shall be provided with an exhaust duct which will draw air from each lamp and exhaust it directly to the outside of the building in such a fashion that it will not be picked up by air supply inlets. Such a duct shall be of rigid materials, except for a continuous flexible connector for the purpose. The lamp exhaust systems shall not be interconnected with any other system.

3.309.16 ELECTRIC ARC PROJECTION EQUIPMENT - The exhaust capacity shall be five and one-half cubic metres (5.5 m^3) per minute for each lamp connected to the lamp exhaust system, or as recommended by the equipment manufacturer. Auxiliary air may be introduced into the system through a screened opening to stabilize the arc.

3.309.17 XENON PROJECTION EQUIPMENT - The lamp exhaust system shall exhaust not less than eight and one-half cubic metres (8.5 m^3) per minute per lamp, nor less than that exhaust volume required or recommended by the equipment manufacturer, whichever is the greater. The external temperature of the lamp housing shall not exceed fifty degrees Celsius (50 degrees C) when operating.

3.309.18 LIGHTING CONTROL (FROM PROJECTION ROOMS) - Provision shall be made for the control of the auditorium lighting, and the emergency lighting systems of theatres from inside of the booth, and from at least one other convenient point elsewhere in the building - all as required under clause 3.318.45 below.

3.309.19 MISCELLANEOUS EQUIPMENT - Each projection room shall be provided with rewind and film storage facilities. A maximum of four (4) containers for flammable liquids not greater than three hundred millilitres (300 ml) capacity and of a non-breakable type may be permitted in each projection booth.

3.309.20 SANITARY FACILITIES - Each projection room shall be provided with a recess (or small room) containing a lavatory basin with trapped waste and running water piped to it. Each projection room serving an assembly occupancy shall also be provided with a water closet.

3.309.21 SCREENING ROOMS - Screening rooms shall provide a seating capacity of not more than thirty (30) persons with not less than two (2) approved means of egress complying with Means of Egress section of this Code (Part 3 Section 5). Such rooms shall be enclosed in one (1) hour fire separation walls with self-closing fire doors or their approved labelled equivalent at the openings. All seats shall be permanently fixed in position and the arrangement shall comply with the requirements of clause 3.315.19 below.

3.309.22 TEMPORARY MOTION PICTURE INSTALLATIONS - Permits for portable and temporary booth construction for incidental amusement and educational purposes shall be secured from the Fire Board/Authority in áccordance with the approved rules.

3.309.23 MOTION PICTURE STUDIOS - CONSTRUCTION - All buildings designed or used as motion picture studios shall be protected with an approved two (2) source automatic sprinkler system complying with the provisions of Part 3 Section 7 of this Code, except that the Building Authority may exempt rooms designed for housing electrical equipment from this requirement when constructed for fireproof (Type 1) construction.

3.309.24 SPECIAL ROOMS - Rooms and spaces used as carpenter and repair shops, dressing rooms, costume and property stage rooms shall be enclosed in floors, walls and ceilings of not less than two (2) hour fire-resistance rated construction.

3.309.25 DECORATIVE HANGINGS FINISH/TRIM - All permanently attached accoustic, insulating, and light reflective materials, and any temporary hangings on walls and/or ceilings, shall comply with the requirements of Part 3 Section 6 of this Code.

3.309.26 CELLULOSE NITRATE FILM STORAGE - All cellulose nitrate film shall be stored as required in clause 3.309.2 and surplus film shall not be kept on the studio stage except loaded magazines in the cameras and sound recording apparatus. All extra loaded magazines shall be stored in a separate magazine room enclosed in two (2) hour fire-resistance rated construction.

3.309.27 FILM LABORATORIES - Film laboratories shall not be located in other than fireproof (Type 1-A) buildings or structures, equipped throughout with an approved automatic sprinkler system.

3.309.28 FILM EXCHANGES - All film exchanges and depots shall be housed in buildings and structures of fireproof (Type 1-A) construction equipped throughout with an approved automatic sprinkler system. All flammable film other than that in process of receipt, delivery or distribution shall be stored in vaults complying with the requirements of clause 3.308.11.

3.310 Use and Storage of Combustible Fibres

3.310.1 GENERAL - The provisions of this sub-section shall apply to all buildings and structures involving the storage or use of finely divided combustible vegetable or animal fibres and thin sheets or flakes of such materials involving flash fire hazard, including among others: cotton, excelsior (shredded paper), hemp, sisal, jute, kapok, and paper and cloth in the form of scraps and clippings in excess of five hundred kilograms (500 kg). The provisions of the applicable standards except as herein specifically provided shall be deemed to conform to the provisions of this part of the Code.

3.310.2 CONSTRUCTION REQUIREMENTS - All buildings designed for the storage of combustible fibres as herein described shall be constructed within the limits of height and area specified in Table 3.205 in Part 3 Section 2 of this Code for high hazard use (occupancy group H) except as described in the following clause 3.310.3 through to and including 3.310.8.

3.310.3 SPECIAL LIMITS - A single storage room or space shall not be more than four hundred and fifty square metres (450 m^2) in area or more than fourteen hundred cubic metres (1400 m^3) in volume unless of protected non-combustible (Type 2-B) or better construction.

3.310.4 FLOOR LOADS - The floors of all buildings designed for the storage of combustible fibres shall not be loaded in excess of one-half (1/2) the safe load capacity of the floor, nor shall such materials be piled to more than two thirds (2/3) of the clear storey height.

3.310.5 SALVAGE DOORS - Every exterior wall shall be provided with a door to each storage compartment arranged for quick removal of the contents.

3.310.6 WALL OPENINGS - All openings in outside walls shall be equipped with approved fire doors and fire windows complying with requirements of Part 3 Section 6 of this Code.

3.310.7 ROOF OPENINGS - All skylights, monitors and other roof openings shall be protected with galvanized wire or other approved corrosion-resistive screens with not less than six (6) meshes to the square centimetre (1.0 cm^2) or with wired glass in stationary frames.

3.310.8 BOILER ROOMS - All power and heating boilers and furnaces shall be located in detached boiler houses or in a segregated boiler room enclosed in three (3) hour fire-resistance rated construction with direct entrance from the outside, except that rooms containing gas-fired heating equipment may have openings into the warehouse protected with one and one-half (1 1/2) hour fire doors or their approved labelled equivalent.

3.310.9 FIRE PROTECTION - First aid fire protection equipment shall be provided complying with Part 3 Section 7 of this Code, consisting of casks, pails, bucket pumps and portable chemical extinguishers as well as standpipes. Where so required or deemed necessary by the administrative (Local) authority, a system of outside hydrants on a ring main with outlets and hoses shall be provided.

3.310.10 HOUSEKEEPING - Ashes, waste, rubbish or sweepings shall not be kept in wood or other combustible receptacles and shall be removed from the premises daily. Grass or weeds shall not be allowed to accumulate at any point on the premises.

3.310.11 OPEN STORAGE - Only temporary open storage or combustible fibres shall be permitted on the same premises with a fibre warehouse and shall be kept covered on top and sides with tarpaulins secured in place. Not more than two hundred (200) cubic metres of fibre shall be stored in the open; and fire-extinguishing equipment shall be provided as directed by the administrative authority.

3.310.12 EXEMPTION FROM RESTRICTIONS (BY USE OF SPECIAL TREATMENT) - When combustible fibres are packed in special non-combustible containers or when packed in bales covered with wrappings to prevent ready ignition, or when treated by approved chemical dipping or spraying processes to eliminate the flash fire hazard, the restrictions governing combustible fibres shall not apply.

3.311 Combustible Dusts, Grain Processing and Storage

3.311.1 GENERAL - The provisions of this sub-section shall apply to all buildings in which materials producing flammable dusts and particles which are readily ignitable and subject to explosion hazards are stored or handled, including, among others, grain bleachers and elevators, malt houses, flour, feed or starch mills, wood flour manufacturing and manufacture and storage of pulverized fuel and similar uses. The applicable standards except as herein specifically required shall be deemed to conform to the requirements of this Code.

3.311.2 CONSTRUCTION REQUIREMENTS (BUILDINGS FOR ABOVE USES) - All such buildings and structures, unless otherwise specifically provided, shall be fireproof (Type 1), non-combustible (Type 2), or of laminated planks or timber sizes qualified for heavy timber mill (Type 3-A) construction, within the height and area limits of high hazard uses (occupancy group H) of Table 3.206 except that when erected of fireproof (Type 1-A) construction, the height and area of grain elevators and similar structures shall be unlimited, and when of heavy timber (Type 3-A) construction, the structure may be erected to a height of twenty metres (20 m); and except further that, in isolated areas, the height of Type 3-A structures may be increased to twenty-five metres (25 m).

3.311.3 GRINDING ROOMS - Every room or space for grinding or other operations producing flammable dust shall be enclosed with floors and walls of not less than two hour fire-resistance rating when the area is not greater than two hundred and fifty square metres (250 m^2), and of not less than four hour fire-resistance rating when the area is greater than two hundred and fifty square metres (250 m^2).

3.311.4 CONVEYORS - All conveyors, chutes, piping and similar equipment passing through the enclosures of such rooms or spaces shall be constructed dirt and vapour tight, of approved non-combustible materials complying with Part 4 Section 3 of this Code (elevators, escalators, dumbwaiters, and conveyor equipment).

3.311.5 EXPLOSION RELIEF - Means of explosion relief shall be provided as specified in sub-section 3.302 above, or such spaces shall be equipped with the eqivalent mechanical ventilation complying with the relevant mechanical Code.

3.311.6 GRAIN ELEVATORS - Grain elevators, malt houses and buildings for similar uses shall not be located within ten metres (10.0 m) of interior boundary lines or structures on the same lot.

3.312 Paint Spraying and Spray Booths

3.312.1 GENERAL - The provisions of this sub-section shall apply to the construction, installation and use of buildings and structures or parts thereof for the spraying of flammable paints, varnishes and lacquers or other flammable materials, mixtures or compounds used for painting, varnishing, staining or similar purpose. All such construction and equipment shall comply with the approved rules and the applicable standards.

3.312.2 LOCATION OF SPRAYING PROCESSES - Such processes shall be conducted in a spraying space, spray booth, spray room or shall be isolated in a detached building or as otherwise approved by the Building Authority in accordance with accepted engineering practice.

3.312.3 CONSTRUCTION (i) OF SPRAY SPACES - All spray spaces shall be ventilated with an approved exhaust system to prevent the accumulation of flammable mist or vapours. When such spaces are not separately enclosed, non-combustible spray curtains shall be provided to restrict the spread of fire.

3.312.4 CONSTRUCTION (ii) OF SPRAY BOOTHS - All spray booths shall be constructed of approved non-combustible materials equipped with mechanical ventilating systems.

3.312.5 CONSTRUCTION (iii) OF SPRAY ROOMS - All spray rooms shall be enclosed in partitions of not less than one (1) hour fire-resistance rating. Floors shall be water-proofed and drained in an approved manner. Floor drains to the building drainage system and the public sewer are prohibited.

3.312.6 STORAGE ROOMS - Spraying materials in quantities of not more than seventy-five litres (75 l) may be stored in approved cabinets ventilated at top and bottom when in quantities of more than seventy-five litres (75 l) but not more than four hundred litres (400 l) they may be stored in approved double-walled non-combustible cabinets vented directly to the outer air; and all spraying materials in quantities of more than four hundred litres (400 l) shall be stored in an enclosure of not less than two (2) hour fire-resistance rating or in a separate exterior storage building. Such storage shall not be equipped in quantities of more than one thousand litres (1000 l) except when stored in isolated storage buildings; and except further that not more than one hundred litres (100 l) of spraying material shall be stored in buildings in which pyroxylin products are manufactured or stored.

3.312.7 VENTILATION OF SPRAYING PROCESSES - The ventilation system shall comply with the provisions of sub-section 3.302 above, and shall be adequate to exhaust all vapours, fumes and residues of spraying material directly to the outer air. Fresh air shall be admitted to the spraying spaces in an amount equal to the capacity of the fan, in such manner as to avoid short-circuiting the path of air in the working space and to provide air movement with a velocity of not less than thirty metres (30 m) per minute at the face of the spray booth. All ducts and vents shall be constructed and installed to comply with the relevant mechanical Code. Unless equipped with approved explosion-proof motors with non-ferrous fan blade fans, the mechanical exhaust equipment shall be located outside of spray spaces.

3.312.8 FIRE PROTECTION (THROUGHOUT SPRAYING AREAS/SPACES) - Sprinkler heads shall be provided in all spray, dip and immersing spaces and storage rooms shall be installed in accordance with accepted engineering practice and relevant standards. When buildings containing spray areas are not equipped with an approved automatic sprinkler system, the sprinkler heads in booths and other spray areas and storage rooms may be supplied from the building water supply when approved by the Building Authority, to comply with the provisions of Part 3 Section 7, sub-section 3.706.

3.313 Dry Cleaning Establishments

3.313.1 GENERAL - Before any dry cleaning plant is constructed or an existing plant is remodelled or altered, complete drawings shall be filed showing to scale the relative location of the dry cleaning area, the boiler room, finishing department, solvent storage tanks, pumps, washers, drying tumblers, extractors, filter traps, stills piping and all other equipment involving the use of flammable liquid solvents. All dry cleaning by immersion and agitation shall be carried on in closed machines, installed and operated in accordance with the approved rules and the applicable standards.

3.313.2 CLASSIFICATION - For the purpose of this Code, all dry cleaning and dry dyeing establishments shall be classified as described in the following clauses 3.313.3 through to and including 3.313.5.

3.313.3 HIGH HAZARD - All such establishments shall be classified as high hazard which employ gasoline or other solvents having a flash point below thirty-eight degrees Celsius (38^{o}C) in quantities of more than twelve litres (12 l), or more than two hundred litres (200 l) of flammable solvents with a flash point between thirty-eight (38) and sixty degrees Celsius (60^{o}C).

3.313.4 MODERATE HAZARD - All such establishments employing less than twelve litres (12 l) of volatile flammables with a flash point of less than thirty-eight degrees Celsius (38 degrees C) or less than two hundred litres (200 l) of solvent with a flash point between thirty-eight (38) and sixty degrees Celsius (60^{o}C) shall be classified as moderate hazard.

3.313.5 LOW HAZARD - All such establishments using solvents of other than volatile flammable liquids or solvents with a flash point more than sixty degrees Celsius (60^{o}C) in cleaning and dyeing operations shall be classified as low hazard.

3.313.6 CONSTRUCTION OF DRY CLEANING PLANTS (i) HIGH HAZARD - High hazard dry cleaning plants as herein defined shall be located in buildings or structures of fireproof (Type 1-A) construction not more than one (1) storey in height with solid floors and roofs and without openings other than required for egress and ventilation purposes. Such building shall not be used for any other purpose.

3.313.7 CONSTRUCTION OF DRY CLEANING PLANTS (i) MODERATE HAZARD - Moderate hazard dry cleaning plants as herein defined may be located in buildings or structures of any type of construction other than frame (Type 4) buildings subject to the fire limit restrictions of Part 3 Section 2 of this Code and the height and area limitations for high hazard buildings (occupancy group H) of Table 3.206. The room or space in which such operations are conducted shall be enclosed in not less than two (2) hour fire-resistance rated construction with not less than two (2) means of egress from each dry cleaning or dry dyeing room or space.

3.313.8 CONSTRUCTION OF DRY CLEANING PLANTS (iii) LOW HAZARD - Low hazard dry cleaning plants shall not be restricted as to type of building construction within the height and area limitations for occupancy group B of Table 3.206; except that such uses shall not be located in basements nor in a building used for public assembly (occupancy group A) or institutional (occupancy group I) purposes.

3.313.9 ROOF CONSTRUCTION OF DRY CLEANING PLANTS - The roof over high hazard dry cleaning plants shall be flat without attic or concealed spaces and shall be provided with a pivot type skylight or other approved vent complying with clauses under 3.302 (in this section of the Code), arranged to release outwardly under explosion pressures.

3.313.10 FLOOR CONSTRUCTION OF DRY CLEANING PLANTS - The floor finish in high hazard dry cleaning plants shall be constructed of water-resistant, non-combustible materials with non-sparking surface elevated above the adjoining levels and with door sills not less that two hundred and fifty millimetres (250 mm) in height. There shall not be openings, vaults, or pits below the floor.

3.313.11 EXTERIOR WALLS OF DRY CLEANING PLANTS - Exterior walls of high hazard dry cleaning plants having a fire separation of less than ten metres (10.0 m) shall be solid masonry without openings, but more than two (2) sides of the building shall not be enclosed in blank walls. Opening protectives of exterior doors and windows shall have not less than three- quarter (3/4) hour fire-resistance or the labelled equivalent construction, and the windows shall be pressure releasing to comply with clauses under 3.302 above.

3.313.12 BASEMENTS OF DRY CLEANING PLANTS - The basements of all buildings in which high or moderate hazard dry cleaning establishments are conducted shall be completely separated from the superstructure with unpierced floor construction of not less than two (2) hours fire-resistance rating. The access to such basements shall be from the exterior only.

3.313.13 BOILER ROOM SEPARATION - Boiler rooms and heating equipment for high hazard dry cleaning plants shall be separated from drying rooms, dry cleaning and dry dyeing rooms with unpierced walls of not less than four (4) hours fire-resistance rating and in moderate hazard establishments with solid walls of not less than two (2) hours fire-resistance rating; or such boiler rooms shall be located in a separate building.

3.313.14 VENTILATION - All rooms and spaces in high hazard dry cleaning plants shall be provided with a mechanical system of ventilation capable of twenty (20) complete and continuous changes of air per hour. Mechanical systems of ventilation in moderate hazard plants shall have sufficient capacity to insure ten (10) complete and continuous changes of air per hour. Satisfactory mechanical or natural ventilation shall be provided in low hazard plants by means of fans, pipes and ducts to ventilate drying tumblers, drying cabinets and similar equipment directly to the outer air.

3.313.15 SOLVENT STORAGE - All volatile flammable solvents with a flash point below twenty-four degrees Celsisus (24°C) shall be stored underground in accordance with the provisions of sub-section 3.303 in this section of the Code. Interior aboveground storage shall be permitted for solvents with a flash point above twenty-four degrees Celsius (24°C), provided the aggregate quantity of such solvent in use in the system and in storage is not more than two thousand litres (2000 l), and the capacity of any individual tank is not more than one thousand litres (1000 l).

3.313.16 FIRE PROTECTION - Every dry cleaning room and dry dyeing room employing high and moderate hazard solvents shall be protected with a fire protection system consisting of approved automatic sprinklers, manually controlled steam-blankets, carbon dioxide flooding systems or other approved fire-extinguishing equipment.

3.314 Private Garages

3.314.1 ATTACHED GARAGES TO ONE AND TWO-FAMILY DWELLINGS - Private garages located beneath or attached to sides, back or front of a one or two-family dwelling shall have walls, partitions, floors and ceilings separating the garage space from the dwelling constructed of not less than one (1) hour fire-resistance rating. Private garages attached to one and two-family dwellings shall be completely separated from the dwelling and its attic area by means of one-half (1/2) inch gypsum board or equivalent applied to the garage side. The sills of all door openings between the garage and dwelling shall be raised not less than one hundred millimetres (100 mm) above the garage floor. The door opening protectives shall be forty-four millimetres (44 mm) thick solid core wood doors or approved equivalent.

3.314.2 MOTELS AND MULTI-FAMILY DWELLINGS - Private garages located beneath motels and multi-family dwellings, and in which gasoline (petrol) or oil is NOT stored or handled shall be of protected construction of not less than one and one-half hour (1 1/2) fire-resistance rating.

3.314.3 SEPARATION BY BREEZEWAY - A garage separated from residence outside the fire limits by a breezeway not less than three metres (3.0 m) in length may be of unprotected frame (Type 4-B) construction, but the junction of the garage and breezeway shall be fire-stopped to comply with clauses under 3.621 in Part 3 Section 6 of this Code.

3.314.4 OTHER CONDITIONS - All private garages not falling within the scope of 3.314.1 to 3.314.3 above attached to, or located beneath a building, shall comply with the requirements of clauses 3.315.5 below for public garages.

3.314.5 HEATING EQUIPMENT - Boilers, furnaces, hot water heaters or any other appliances having an open flame or exposed heated surfaces shall not be located in a private garage unless precautions are taken to protect such equipment from impact by automobiles. This equipment shall have the combustion chamber, ash pit etc., raised a minimum of one half of one metre (0.5 m) above the floor to eliminate a possible source of ignition.

3.314.6 MEANS OF EGRESS - Where living quarters are located above a private garage, required means of egress facilities shall be protected from the garage area with one (1) hour fire-resistance rated construction.

3.315 Public Garages

3.315.1 GENERAL - Public garages shall comply with the applicable requirements of this section. The portions of such buildings and structures in which gasoline, oil and similar products are dispensed shall comply with the requirements of clauses under 3.316 below; the portions in which motor vehicles are repaired shall comply with clauses under 3.317; and the portions in which paint spraying is done shall comply with the requirements of clauses under 3.313 above.

3.315.2 CONSTRUCTION - All Group I public garages hereafter erected shall be classified as storage buildings, moderate hazard (occupancy group S-1) and all Group 2 public garages shall be classified as storage buildings, low hazard (occupancy group S-2) and shall conform to the height and area limitations of Table 3.206 (Part 3 Section 2 of this Code) except as herein specifically provided. The areas used for dispensing gasoline in such buildings shall be located on the ground floor and shall comply with the requirements of clauses under 3.316 below.

3.315.3 SPECIAL HEIGHT LIMITATIONS - Public garage buildings shall comply with the height and area limitations of Table 3.206 (Part 3 Section 2) for the classification of the use as specified in clause 3.315.2 above. Such heights may be increased one (1) additional storey when the building is equipped with an approved automatic fire suppression system.

3.315.4 BASEMENTS - The ground floor construction of public garages of all classifications (and public hangars) with basements shall be constructed of not less than two (2) hour fire-resistance rating and shall be water and vapour-proof. Where openings are provided in the floor they shall be protected by a curb or ramp not less than one hundred and fifty millimetres (150 mm) high above the floor to avoid the accumulation of explosive liquids or vapours and prevent them from spilling to the lower floor. There shall be not less than two (2) means of egress from such areas, one (1) of which shall be directly to the outside independent of the exitways serving other areas of the building.

3.315.5 MIXED OCCUPANCY - A public garage shall not be located within or attached to a building occupied for any other use, unless separated from such other use by walls or floors complying with Table 3.604 (in Part 3 Section 6 of this Code) for fire-resistance rating. Such fire separation walls shall be continuous and unpierced by openings; except that door openings equipped with self-closing fire doors all complying with the requirements under Part 3 Section 6 of this Code shall be permitted. In buildings of single occupancy not excluding the area limitations of Table 3.206, doors without a fire-resistance rating shall be permitted between the garage area and salesroom or offices that are operated in connection with the garage.

3.315.6 ROOF PARKING OR STORAGE OF MOTOR VEHICLES (AND AIRPLANES OR HELICOPTERS) - The roof of a public garage shall not be used for the parking or storage of motor vehicles unless the building is of fireproof construction (Type 1-A or 1-B). When the roof of a building is used for parking or storage of motor vehicles it shall be provided with a parapet wall or guard rail not less than one metre (1.0 m) in height and a wheel guard not less than one hundred and fifty millimetres in height, located so as to prevent any vehicle from striking the parapet wall or guard rail. The use of roofs for airplane/helicopter storage and landing shall be subject to the approval of the local aviation administration if required.

3.315.7 FLOOR CONSTRUCTION AND DRAINAGE - Floors of public garages and airplane hangars shall be graded to drain through oil separators or traps to avoid accumulation of explosive vapours in building drains or sewers as provided in the Plumbing Code. The floor finish shall be of concrete or other approved non-absorbent, non-combustible material.

3.315.8 VENTILATION - All public garages and airplane hangars shall be provided with mechanical or natural ventilation adequate to prevent the accumulation of carbon monoxide or exhaust fumes in excess of one (1) part in ten thousand (10,000) or one-tenth of one percent (.01%) or the concentration of gasoline vapours in excess of twenty (20) percent of the lower explosive limit. The Building Authority may require a test by a qualified testing laboratory to determine the adequacy. The cost of test shall be borne by the owner.

3.315.9 BELOW GROUND GARAGES - Enclosed, partly below ground, or basement public garages shall be equipped with mechanical ventilation adequate to provide six (6) air changes per hour. The ventilation system shall be operated at all times the garage areas are occupied (by people).

3.315.10 REPAIR SHOPS OR ROOMS - When motor vehicles are to be operated, or engines are run for test purposes, or minor adjustments, provisions shall be made to collect the exhaust fumes from each vehicle individually and to discharge such fumes to the outer air by means of positive induced draft. The discharge from such a system shall be located so as not to create a hazard to adjoining properties, but not less than two and one-half metres (2.5 m) above the adjacent ground level on the exterior of the building and shall discharge into a yard or court. When necessary to discharge across a walkway or private thoroughfare, the discharge opening shall be carried to a height of not less than seven and one-half metres (7.5 m) above the ground level or to a distance of one hundred millimetres (100 mm) above the highest point of the wall of the building or structure on which it is located.

3.315.11 PITS - Pits shall not be installed in floors below ground floors, and pits in ground and any upper stories shall be provided with mechanical ventilation adequate to provide the ventilation required under clause 3.315.8 above. The ventilation system shall be operated at all times the pits are occupied by workmen or other personnel.

3.315.12 SPECIAL HAZARDS - Any process conducted in conjunction with public garages involving volatile flammable solvents or battery charging or similar shall be segregated or located in a detached building or structure, except as provided in clauses under 3.303 above for the storage and handling of gasoline and other volatile flammables. The quantity of flammable liquids stored or handled in public garages other than in underground storage and in the tanks of motor vehicles shall be not more than twenty litres (20 l) in approved safety cans.

3.315.13 BOILER ROOMS OF PUBLIC GARAGES - All heat generating plants other than approved direct fired heaters shall be located in separate buildings or shall be separately enclosed within the structure with solid, water and vapour-tight masonry. All rooms housing boilers or stoves or other heating apparatus shall be cut off from all other parts of the building with four (4) hour fire-resistance rated construction with entrance from outside only, and there shall not be openings through the fire separation wall other than those necessary for pipes and ducts.

3.316 Motor Fuel Service Stations

3.316.1 CONSTRUCTION - Buildings and structures used for the storage and sale of motor fuel oils may be all types of construction within the height and area limitations of Table 3.206 for business (occupancy group B) buildings and as modified by Part 3 Section 2 sub-section 3.204. The canopies and supports over pumps and service equipment when located less than six metres (6.0 m) from interior boundary lines shall be constructed of approved non-combustible materials, Type 3-A (heavy timber) construction, or one (1) hour fire-resistance rated construction.

3.316.2 EXCEPTIONS - Approved plastics (see also under Part 4 Section 5 of this Code) may be used as canopies over pumps when conforming to the following requirements:

(i) The canopies are located at least three metres (3.0 m) from any building on the same property and face yards or streets not less than twelve metres (12 m) wide on the other sides, or

(ii) The aggregate area of plastic in each canopy shall not exceed twenty square metres (20 m^2) in the fire limits, (i.e. in central city areas) or one hundred square metres (100 m^2) outside the fire limits and

(iii) The maximum area of each panel shall not exceed nine square metres (9 m^2).

3.316.3 OPENING PROTECTIVES - All permissible openings in walls with a fire separation of less than six metres (6.0 m) shall be protected with approved fire windows or fire doors complying with Part 3 Section 6 of this Code except for doors in such walls giving access to rest rooms.

3.316.4 BASEMENTS - Motor fuel service stations shall not have cellars or basements and when pits are provided they shall be vented as required in clause 3.315.11 above.

3.316.5 GASOLINE STORAGE - All volatile flammable liquid storage tanks shall be installed below ground and vented as specified in clauses under 3.303 above. Gasoline may be stored or handled above ground in approved safety cans of not more than twenty litres (20 l) each.

3.316.6 LOCATION OF PUMPS - Gasoline pumps or other mechanical equipment shall not be installed so as to permit servicing of motor vehicles standing on a public street or highway except when necessitated by the widening of streets or highways; the use of the outer driveway of existing service stations may be continued for servicing of vehicles when approved by the authority having jurisdiction.

3.317 Motor Vehicle Repair Shops

3.317.1 GENERAL - All buildings or shelters designed and used for the repair and servicing of motor vehicles, motor boats, tractors, airplanes or other automotive machinery or means of transportation shall be subject to the limitations of Table 3.206 (Part 3 Section 2 of this Code) and for moderate hazard storage (occupancy group S-1). Such buildings shall be used solely for that purpose.

3.317.2 ENCLOSURE WALLS - Exterior walls, when located within two metres (2.0 m) of interior boundary lines or other buildings shall not have openings therein.

3.317.3 HANDLING OF VOLATILE FLAMMABLES - All volatile flammables shall be stored and handled as provided in clause 3.316.5 above.

3.317.4 VENTILATION - All rooms and spaces used for motor vehicle repair shop purposes shall be provided with mechanical or natural ventilation adequate to prevent the accumulation of carbon monoxide or exhaust fumes or other injurious gases in excess of one part in ten thousand (1 in 10,000) or one-tenth of one percent 0.01% or the concentration of gasoline vapours in excess of twenty percent (20%) of the lower explosive limit - see also clause 3.315.8 above re percentages and tests.

3.317.5 FIRE PREVENTION - Open gas flames (except heating devices complying with clause 3.315.13) torches, welding apparatus, or other equipment likely to create an open flame or spark shall not be located in a room or space in which flammable liquids or highly combustible materials are used or stored.

3.318 Places of Public Assembly

3.318.1 APPLICABILITY - The provisions of this sub-section shall apply to all places of public assembly and all parts of buildings and structures classified in the occupancy group A-1, theatres and in other places of public assembly, occupancy groups A-2, A-3, and A-4, except as specifically exempted in 3.319 below.

3.318.2 RESTRICTIONS - The following restrictions as outlined in 3.318.3 to and including 3.318.9 shall apply to places of public assembly.

3.318.3 HIGH HAZARD USES - A place of public assembly shall not be permitted in a building classified in the high hazard group, (occupancy group H).

3.318.4 SUPERIMPOSED THEATRES - An addition or extension shall not be erected over the stage section of a theatre, nor shall a second theatre be erected above another. The Building Authority may waive the prohibition against super-imposed theatres and construction above the stage when adequate access is provided for fire-fighting with direct means of ventilation to the outer air from the stage portion.

3.318.5 FRAME CONSTRUCTION - A theatre with stage, fly gallery and rigging loft shall not be permitted in a building of unprotected frame (Type 4-B) construction.

3.318.6 LOCATION - All buildings used for assembly purposes shall front on at least one (1) street in which the main entrance and exitway discharge shall be located. The total capacity of such main exitway shall be not less than one-third (1/3) of the total required width of building exitways.

3.318.7 TRIM, FINISH AND DECORATIVE HANGINGS - All permanent accoustic insulating and similar materials and temporary hangings shall comply with the flame-resistance requirements of Part 3 Section 6 of this Code. Moldings and decorations around the proscenium openings shall be constructed entirely of non-combustible material.

3.318.8 EXISTING BUILDINGS - Nothing herein contained shall prohibit the alteration of a building heretofore occupied as a place of public assembly for such continued use provided the occupancy load is not increased and seats, aisles, passageways, balconies, stages, appurtenant rooms and all special permanent equipment comply with the requirements of this article.

3.318.9 NEW BUILDINGS - A building not previously occupied as a place of public assembly, shall not hereafter be altered to be so occupied unless it is made to comply with all the provisions of this sub-section.

3.318.10 THEATRE MEANS OF EGRESS REQUIREMENTS - Although Part 3 Section 5 of this Code covers Means of Egress from all types of buildings and occupancies, this clause 3.318.10 up to and including 3.318.18 deals exclusively with theatres; it must be read in conjunction with Part 3 Section 5, and should there be any apparent conflict in requirements, the more stringent of the two shall be accepted.

3.318.11 TYPES OF EXITWAYS - The required exitways from every tier or floor of a theatre shall consist of ground level exitway discharge doors, interior or exterior stairways or horizontal exits which provide direct access to a street, an exitway discharge court, or unobstructed passageway, hallway or lobby leading to a street or open public space. The number, location and construction of all means of egress facilities shall comply with the requirements of Part 3 Section 5 of this Code and other applicable standards, except as herein specifically provided.

3.318.12 NUMBER OF STAIRWAYS IN AUDITORIUM - Each tier above the main floor of a theatre or other auditorium shall be provided with at least two (2) interior enclosed stairways which shall be located on opposite sides of the structure; except that enclosures shall not be required for stairs serving the first balcony only, or mezzanine thereunder. Such stairways shall discharge to a lobby on the main floor.

3.318.13 EMERGENCY MEANS OF EGRESS FROM MAIN FLOOR AUDITORIUM - In addition to the main floor entrance and exitway, emergency exitway discharge doors shall be provided on both sides of the auditorium which lead directly to a street, or through a passageway to the street independent of other exitways, or to an exitway discharge court as defined in this Code.

3.318.14 EMERGENCY MEANS OF EGRESS FROM BALCONIES AND GALLERIES - Emergency exitways shall be provided from both sides of each balcony and gallery with direct egress to the street, or to an independent passage-way, or to an exitway discharge court. There shall not be communication from any portion of the building to the emergency exitway stairways except from the tier for which such exitway is exclusively intended.

3.318.15 EXITWAY DISCHARGE COURTS - All exitway discharge courts shall be not less than two metres (2.0 m) wide for the first six hundred (600) persons to be accommodated or fraction thereof, and shall be increased three hundred millimetres (300 mm = 0.3 m) width for each additional two hundred and fifty (250) persons. Such courts shall extend sufficiently in length to include the side and rear emergency exitways from the auditorium.

3.318.16 WIDTH OF EXITWAY DOORS - The maximum width of single exitway doors shall be one and one-tenth metres (1.1 m) and the minimum width of double doorways shall be one and eighty-five hundredths of one metre (1.85 m).

Note: It should be noted that although the maximum width allowable for single exit doors given in clause 3.514.4 (Part 3 Section 5 of this Code) is one and one-fifth metres (1.2 m) the maximum width for a single exit door from a theatre main floor auditorium is slightly less (one and one-tenth metres) and this must be strictly adhered to.

3.318.17 HARDWARE - Latches or bolts on all means of egress doorways shall be of an approved self-releasing, panic-proof type complying with clause 3.514.8 (in Part 3 Section 5 of this Code).

3.318.18 "EXIT" LIGHTS - All exitway doors shall be marked with illuminated "Exit" signs complying with clauses 3.523.1 and 3.523.2 (in Part 3 Section 5 of this Code).

3.318.19 THEATRE SEATING - The various clauses of this sub-section also apply to smaller "screening rooms" - see above under 3.309.21.

3.318.20 FIXED SEATS - In all theatres and similar places of assembly except churches, stadiums and reviewing stands, individual fixed seats shall be provided with an average width of not less than five hundred and ten millimetres (510 mm) and seats shall not be less than four hundred and eighty-five millimetres (485 mm) wide. All seats shall be provided with separating arms and arranged in rows not less than eight hundred and fifteen millmetres (815 mm) apart, back to back, measured horizontally.

3.318.21 NUMBER OF SEATS - Aisles shall be provided so that not more than six (6) seats intervene between any seat and the aisle or aisles, except that the number of seats in a row shall not be limited when self-raising seats are provided which leave an unobstructed passage between rows of seats of not less than four hundred and fifty millimetres (450 mm) in width leading to side aisles in which exitway doorways are located at not more than seven and one-half metres (7.5 m) intervals to the exitway corridor or exitway discharge court.

3.318.22 BOX SEATS - In boxes or "loges" with level floors, the seats need not be fastened down when not more than fourteen (14) in number.

3.318.23 THEATRE AISLES - LONGITUDINAL - The width of longitudinal aisles at right angles to rows of seats and with seats on both sides of the aisle shall be not less than one metre (1.0 m), increasing two centimetres in width (2cm) for every one metre (1.0 m) of length of aisle from its beginning to an exitway door, or to a cross aisle or between cross aisles. The width of the longitudinal aisles with banks of seats on one side only shall be not less than seven hundred and sixty millimetres (760 mm or .76 m), increasing as above by two centimetres for every metre (1.0 m) of length.

3.318.24 THEATRE AISLES - ACROSS - When there are twenty seven (27) or more rows of seats on the main floor of theatres, cross aisles shall be provided so that a block of seats shall not have more than twenty-two (22) rows. The width of cross aisles shall be not less than the widest aisle with which they connect or the width of exitway which they serve; but a cross aisle shall not be less than one and one-tenth metres (1.1 m) wide, or when bordering on means of entrance not less than one and one-fifth metres (1.2 m) wide. In balconies and galleries of theatres, one (1) or more cross aisles shall be provided when there are more than ten (10) rows of seats.

3.318.25 GRADIENT - Aisles shall not exceed a gradient of one in seven (1 in 7) i.e. one metre fall (or rise) in seven metres (7.0 m) of length of aisle.

3.318.26 BALCONY STEPS - Steps may be provided in balconies and galleries only, and such steps shall extend the full width of the aisle with treads and risers complying with Part 3, Section 5 of this Code which shall be illuminated by lights on both sides or by a step light or otherwise to insure an intensity of not less than one-tenth of one lux (0.1 lx).

3.318.27 RAILINGS - Metal or other approved non-combustible railings shall be provided on balconies and galleries as prescribed below.

(i) At the fascia of boxes, balconies and galleries not less than seven hundred and fifty millimetres (750 mm) in height; and not less than nine hundred and ten millimetres (910 mm) in height at the foot of steps;

(ii) along cross aisles not less than five hundred and twenty millimetres (520 mm) in height except where the backs of the seats along the front of the aisle project five hundred and twenty millimetres (520 mm); and

(iii) where seatings are arranged in successive tiers, and the height of rise between platforms exceeds four hundred and fifty millimetres (450 mm) not less than five hundred and twenty millimetres (520 mm) in height along the entire row of seats at the edge of the platform.

3.318.28 THEATRE FOYERS - CAPACITY OF - In every theatre or similar place of public assembly, (excluding churches) for theatrical use with stage and scenery loft, a foyer or lobby shall be provided with a net floor area, exclusive of stairs or landings, of not less than one-seventh of one square metre (0.14 m^2) for each occupant having access thereto. The use of foyers and lobbies and other available spaces for harboring occupants until seats become available shall not encroach upon the clear floor area herein prescribed or upon the required clear width of front exitways.

3.318.29 THEATRE FOYERS - EGRESS FROM - When the foyer is not directly connected to the public street through the main lobby, an unobstructed corridor or passage shall be provided which leads to and equals the required minimum width of main entrances and exitways. A mirror shall not be placed so as to give an appearance as a doorway, exit or passageway.

3.318.30 FOYER LEVEL AND EGRESS GRADIENT - The rear foyer shall be at the same level as the back of the auditorium and the means of egress leading therefrom shall not have a steeper gradient than one in eight (i.e. one hundred and twenty-five millimetres (125 mm) rise or fall in one metre (1.0 m) of length).

3.318.31 CONSTRUCTION OF FOYER WALLS - The partitions separating the foyer from the auditorium and other adjoining rooms and spaces of theatres shall be constructed of not less than two (2) hour fire-resistance rating; except that opening protectives may be constructed of non-combustible materials without fire-resistance rating.

3.318.32 WAITING SPACES - Waiting spaces for harbouring occupants shall be located only on the ground or auditorium floor. Separate exitways in addition to the required theatre exitways shall be provided from the waiting space based on an occupancy of three (3) persons for each square metre (1.0 m^2) of waiting area.

3.318.33 THEATRE STAGE CONSTRUCTION (i) WALLS - Every stage hereafter erected or altered for theatrical performances which is equipped with portable or fixed scenery, lights and mechanical appliances shall be enclosed on all sides with solid walls of not less than four (4) hour fire-resistance rating, extending continuously from the foundation to at least one and one-fifth metres (1.2 m) above the roof. There shall not be window openings in such walls within two metres (2.0 m) of an interior boundary/lot line; and all permissible window openings shall be protected with three-quarter (3/4) hour fire windows complying with Part 3 Section 6 of this Code.

3.318.34 STAGE: FLOOR CONSTRUCTION (ii) - The entire stage, except that portion used for the working of scenery, traps, and other mechanical apparatus for the presentation of a scene, and the roof over the stage shall be not less than three (3) hour fire-resistance rated construction. All openings through the stage floor shall be equipped with tight fitting, solid wood trap doors not less than seventy-five millimetres (75 mm) in thickness or other materials of equal physical and fire-resistance rated properties.

3.318.35 STAGE - RIGGING LOFT (iii) - The rigging loft, fly galleries and pin rails shall be constructed of approved non-combustible materials.

3.318.36 STAGE FOOTLIGHTS AND ELECTRICAL EQUIPMENT (iv) - Footlights and border lights shall be installed in troughs constructed of non-combustible materials. The switchboard shall be so located as to be readily accessible at all times and the storage or placing of stage equipment against it shall be prohibited.

3.318.37 EXTERIOR DOORS OFF STAGE - All required exitway discharge door openings to the outer air shall be protected with approved self-closing fire doors, complying with Part 3 Section 6 of this Code. All exterior openings which are located on the stage for means of egress or loading and unloading purposes which are likely to open during occupancy of the theatre, shall be constructed with vestibules to prevent air draughts into the auditorium.

3.318.38 PROSCENIUM WALL - There shall not be other openings in the wall separating the stage from the auditorium except the main proscenium opening; two (2) doorways at the stage level, one (1) on each side thereof; and, where necessary, not more than two (2) doorways to the musicians' pit from the space below the stage floor. Each such doorway shall not exceed two square metres (2 m^2) in area and shall be protected with approved automatic and self-closing fire door assemblies complying with Part 3 Section 6 of this Code with a combined fire-resistance rating of three (3) hours or the approved labelled equivalent. The distance between the top of the proscenium opening and the ceiling of the stage shall be not less than one and one-half metres (1.5 m).

3.318.39 PROSCENIUM CURTAIN - The proscenium opening shall be protected with an automatic fire-resistive and smoke-tight curtain designed to resist an air pressure of not less than nine and nine-tenths megapascals (9.9 M Pa) normal to its surface, both inward and outward. The curtain shall withstand a one-half (1/2) hour fire test at a temperature of not less than nine hundred and twenty-five degrees Celsius (925°C) without the passage of flame. The curtain shall be operated by an automatic heat activated device to descend instantly and safely and to completely close the proscenium opening at a rate of temperature rise of between eight (8) and eleven (11) degrees Celsius per minute and by an auxiliary operating device to permit prompt and immediate manual closing of the proscenium opening.

3.318.40 SCENERY - All combustible materials used in sets and scenery shall be rendered flame-resistant to comply with Part 3 Section 6 of this Code.

3.318.41 STAGE VENTILATION - Metal or approved non-combustible ventilators, equipped with movable shutters or sash shall be provided over the stage, constructed to open automatically and instantly by approved heat activated devices, with an aggregate clear area of opening not less than one-eighth (1/8) the area of the stage, except as otherwise provided in clause 3.318.4 above. Supplemental means shall be provided for manual operation of the ventilator.

3.318.42 DRESSING AND APPURTENANT ROOMS - CONSTRUCTION OF - Dressing rooms, scene docks, property rooms, work shops and store rooms and all compartments appurtenant to the stage shall be of fire-proof (Type 1) construction and shall be separated from the stage and all other parts of the building by walls of not less than three (3) hour fire-resistance rating. Such rooms shall not be placed immediately over or under the operating stage area.

3.318.43 OPENING PROTECTIVES - Openings other than to trunk rooms and the necessary doorways at stage level shall not connect such rooms with the stage, and such openings shall be protected with one and one-half (1 1/2) hour self-closing fire doors or the approved labelled equivalent complying with Part 3 Section 6.

3.318.44 DRESSING ROOM AND STAGE EXITWAYS - Each tier of dressing rooms shall be provided with at least two means of egress; one of which shall lead directly to an exitway corridor, exitway discharge court or a street. Exitway stairways from dressing and storage rooms may be unenclosed in the stage area behind the proscenium wall. At least one approved exitway shall be provided from each side of the stage and from each side of the space under the stage, and from each fly-gallery to a street, exitway discharge court of passageway to a street. An iron ladder shall be provided from the gridiron to a scuttle in the stage roof.

3.318.45 LIGHTING (i) EXITWAYS - During occupancy all exitways in places of assembly shall be lighted to comply with the requirements of Part 3 Section 5 of this Code (sub-section 3.524).

3.318.46 LIGHTING (ii) AUDITORIUMS - Aisles in auditoriums shall be provided with general illumination of not less than one-fifth of one lux (0.2 lx) at the front row of seats and also at the last row of seats and illumination shall be maintained throughout the performance, but during the showing of motion pictures or other projections this can be reduced to one-fiftieth of one lux (0.02 lx) see clause 3.524.2) (in Part 3 Section 5 of this Code) provided that the level of illuminance is immediately and automatically increased to one-fifth of one lux (0.2 lx) in the event of a fire or any other emergency.

3.318.47 LIGHTING (iii) OTHER PLACES OF PUBLIC ASSEMBLY - All areas and portions of buildings used as places of public assembly other than theatres shall be lighted by electric light to provide a general illumination of not less than one-fifth of one lux (0.2 lx).

3.318.48 CONTROL - The lighting of exitways, aisles and auditoriums shall be controlled from a location inaccessible to unauthorized persons. Supplementary control shall be provided as specified in clause 3.309.18 above in the motion picture projection room.

3.318.49 FIRE PROTECTION AND FIRE-FIGHTING EQUIPMENT - Every theatre classified in the use Group A-1 shall be equipped with a fire-protection system complying with the requirements of Part 3, Section 7 of this Code and as herein specified.

3.318.50 FIRE SUPPRESSION SYSTEM - Approved automatic fire suppression systems complying with the provisions of sub-section 3.703 and onwards in Part 3 Section 7 of this Code shall be provided to protect all parts of the building except the auditorium, foyers and lobbies or in the immediate vicinity of automatic equipment or over dynamos and electric equipment. Such protection shall be provided over the stage, under the gridiron, under all fly-galleries in dressing rooms, over the proscenium openings on the stage side, under the stage, in all basements, cellars, work rooms, store rooms, property rooms and in toilet, lounge and smoking rooms.

3.318.51 STANDPIPES - Standpipe fire-fighting mains complying with the provisions of sub-section 3.712 (Part 3 Section 7) shall be provided with outlets and hose attachments, one (1) on each side of the auditorium in each tier, one (1) in each mezzanine, one (1) in each tier of dressing rooms; and protecting each property, store and work room.

3.318.52 FIRST AID STANDPIPES - First aid standpipes complying with the provisions of sub-section 3.712 (as above) shall be provided on each side of the stage. Such standpipes shall not be less than sixty-five millimetres (65 mm) in diameter, and equipped with thirty-eight millimetre (38 mm) hose and ten millimetre (10 mm) nozzles.

3.318.53 FIRST AID HAND EQUIPMENT - Approved portable ten litre (10 l) fire extinguishers shall be provided and located as follows: two (2) on each tier or floor of the stage; one (1) in each dressing room; and one (1) in each work, utility and storage room. Fire axes and fire hooks shall also be provided as directed by the fire authority; and all fire extinguishers and fire tools shall be securely mounted on walls in plain view and readily accessible.

3.319 **Public Assembly Other than Theatres**

3.319.1 GENERAL - Other places of public assembly, including auditoriums, armories, bowling alleys, broadcasting studios, chapels, churches, community houses, dance halls, gymnasiums, lecture halls, museums, exhibition halls, night clubs, rinks, roof gardens and similar occupancies and uses shall comply with the general exitway requirements of Part 3 Section 5 of this Code and the applicable requirements of clauses under 3.318 above, except the provisions of clauses 3.318.27 and 3.318.32 or as herein specifically exempted. Places of public assembly which are equipped with a stage, movable scenery, scenery loft and dressing rooms shall comply with all the requirements of clauses under 3.318 except occupancy group A-1, theatres.

3.319.2 NUMBER OF EXITWAYS - Every tier, floor level and storey of places of public assembly other than theatres, shall be provided with the number of required exitways as specified in Table 3.319 and clauses 3.511.2 (in Part 3 Section 5) and of not less than the required width complying with clauses under 3.510 (Part 3 Section 5) for the occupancy load. The required exitways shall be remote and independent of each other and located on opposite sides of the area served thereby.

TABLE 3.319

MINIMUM NUMBER OF EXITWAYS FOR OCCUPANCY LOADS

Occupancy Load per Floor	Minimum Number of Exitways
Not more than 500"	2
501 to 900	3
901 to 1800	4
over 1800	5

3.319.3 AISLES WITH FIXED SEATS - All rows of seats shall be individually fixed or fixed in rigid units between longitudinal aisles complying with clauses 3.318.20 and 3.318.23 above except as provided for chapels and churches in clause 3.512.5 (Part 3 Section 5 of this Code). Where permitted, continuous fixed benches shall comply with subsection 3.322 below, in Section 3A.

3.319.4 AISLES WITHOUT FIXED SEATS - Tables and chairs in all rooms and spaces for public assembly shall provide convenient access by unobstructed aisles not less than one metre (1.0 m) wide which lead to required exitways complying with (Part 3 Section 5 of this Code).

CARIBBEAN UNIFORM BUILDING CODE

PART 3
OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 3A

SPECIAL USE AND OCCUPANCY REQUIREMENTS FOR HIGH HAZARD 'OPEN AIR' TYPE OCCUPANCIES

PART 3

SECTION 3A
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PART 3

Section 3A (following on from Section 3)

3.300A SPECIAL USE AND OCCUPANCY REQUIREMENTS FOR HIGH HAZARD "OPEN AIR" TYPE OCCUPANCIES

3.320 Scope

3.320.1 As this section is a continuation of Section 3, scope is the same as covered in clause 3.301.1 above.

3.320.2 This section (3A), although really a continuation of 3 is further sub-divided into smaller sub-sections (a) to (n) as follows:

3.321 Amusement Parks

3.321.1 CONSTRUCTION - All accessory buildings and enclosed structures shall be constructed to conform to the requirements of this Code governing use and occupancy as regulated by Table 3.114 (Part 3 Section 1 of this Code) and in compliance with the fire limit restrictions of Part 3 Section 2 of this Code, except as may be specifically required in the following clauses 3.321.2 to and including 3.321.4.

3.321.2 AMUSEMENT DEVICES - The maximum height of any amusement device in which passengers are transported shall not exceed twelve metres (12 m) in frame (Type 4) construction; thirty metres (30 m) in unprotected non-cumbustible (Type 2C) and heavy timber mill (Type 3-A) construction; and shall not be limited in fire-proof (Type 1) construction.

3.321.3 AMUSEMENT PARK BUILDINGS - All enclosed amusement park buildings over one (1) storey in height shall be constructed or protected to furnish not less than one (1) hour fire-resistance rating except where roof framing and decking are specifically permitted to be of non-combustible or mill type construction under the provisions of this Code.

3.321.4 PROXIMITY TO BOUNDARIES (LOT LINES) OR OTHER BUILDINGS - All structures located within six metres (6 m) of boundaries of parks or amusement areas or other buildings on the same section or lot shall be protected non-combustible (Type 2-B) or protected masonry enclosed (Type 3-A or 3-B) construction or better.

3.321.5 WALKWAYS AND RAMPS - Walkways and ramps shall be erected with a slope not greater than one (1) in ten (10), except that when approved non-slip surfaces are provided, the grade may be increased to a maximum of one (1) in eight (8).

3.321.6 ELEVATING AND CONVEYING EQUIPMENT - The equipment and operation of all devices and mechanisms for transporting persons shall comply with the requirements of Part 4 Section 3 of this Code.

3.321.7 TESTS - All amusement devices used by the public which involve hazardous features shall be installed and operated as directed by the building authority and shall not be placed in service until acceptance tests have been made and the installation has been approved by the relevant inspector for that body.

3.321.8 FIRE PROTECTION - In addition to the fire extinguisher and fire-fighting equipment required by the use and occupancy of each building and structure under the provisions of this Code, every amusement and exhibition park, when required by the building authority shall be provided with a system of fire hydrants and fire lines with the required water supply, complying with Part 3 Section 7 of this Code and other relevant standards.

3.322 Stadiums and Grandstands

3.322.1 GENERAL - Stadiums and grandstands shall be constructed as required by this Code and in accordance with the approved rules and the Standard for Tents, Grandstands and Air-Supported Structures used for Places of Assembly (NFiPA 102) and/or other relevant standards.

3.322.2 HANDRAILS - Means of egress stairways shall be provided with a handrail on at least one (1) side. The handrail may be broken as necessary to provide for entrance to the seating platforms.

3.322.3 SPACES UNDERNEATH SEATS - Spaces underneath grandstand seats shall be kept free of all combustible and flammable materials and shall not be occupied or used for other than exitways; except that when enclosed in not less than one (1) hour fire-resistance rated construction, the Building Authority may approve the use of such spaces for other purposes that do not endanger the safety to public.

3.323 Drive-In Motion Picture Theatres

3.323.1 LOCATION - The location of drive-in motion picture theatres shall be approved by the Local or State Authority having jurisdiction over highways and streets.

3.323.2 ARRANGEMENT OF LANES - Separate entrance and exit lanes shall be provided not less than three and three quarter metres (3.75 m) in width, with not less than twelve metres (12 m) intervals between access lanes. The parking space for each car shall not be less than three metres (3.0 m) by six metres (6.0 m) in area, and so arranged to provide continuous lanes of travel.

3.323.3 PROJECTION BOOTH - The projection booth shall comply with clauses under 3.309.9 above and shall be supported on a structure of Type 2-C or other approved non-combustible construction. A motor vehicle shall not be permitted to park within six metres (6 m) of the projection booth or room.

3.323.4 TOILET FACILITIES - Separate toilet facilities shall be provided for each sex as required in the plumbing Code for places of public assembly.

3.323.5 FIRE PROTECTION - Sufficient approved portable fire extinguishers shall be provided in readily accessible locations, plainly and visibly identified by signs, at distances of not more than forty-five metres (45 m) so as to be available to every motor vehicle as directed by the Fire Authority. The fire extinguishers shall be mounted on posts or platforms protected from mechanical injury with substantial guards as approved by the Building Authority.

3.324 Tents, Air-Supported Structures, and Other Temporary Structures

3.324.1 CONSTRUCTION - Tents and air-supported structures shall be constructed as required by this Code and the approved rules.

3.324.2 PERMITS - A special temporary permit shall be secured from the Building Authority for all such installations. Tents, air-supported structures and other temporary structures may be erected for a period not exceeding ninety (90) days for religious, educational, recreational, or similar purposes.

3.324.3 LOCATION - Tents and air-supported structures shall be located outside the fire limits unless an accessible unoccupied open space is provided around the perimeter with a minimum width of fifteen metres (15 m).

3.324.4 APPROVED TYPE - Tents, air-supported structures and other temporary structures shall be of an approved type. The applicant for a special temporary permit hereunder shall submit evidence of the adequacy of the temporary structure in accordance with the requirements of Part 1 Section 1 of this Code.

3.324.5 FIRE PREVENTION - REMOVAL OF RUBBISH - Combustible materials shall not be permitted under stand or seats at any time.

3.324.6 FLAMMABLE RUBBISH - (COMBUSTIBLE TRASH) - The area within and adjacent to tents, air-supported structures or other temporary structures shall be maintained clear of all grass or underbrush creating a fire hazard within a radius of fifteen metres (15 m) and all combustible trash shall be removed from the structure after each performance.

3.325 Car Parks - Parking Lots

3.325.1 CURBCUTS - All car parks shall be arranged to afford ready means of entrance and exit at sidewalk (footpath) level; and special permits shall be secured for curbcuts or vehicle crossings from the administrative authorities.

3.325.2 LANES AND PARKING SPACES - Access lanes not less than three and three-quarter metres (3.75 m) in width shall be provided for each row of cars; and the parking space shall be not less than two and one-half metres (2.5 m) by five and one-half metres (5.5 m) in area for each motor vehicle.

3.325.3 CAR PARK OFFICES - The construction of car park offices shall comply with the fire limit restrictions of Part 3 Section 2 (clauses under 3.203).

3.325.4 PROTECTION OF ADJOINING PROPERTY - A substantial bumper of masonry, steel or heavy timber shall be placed near all interior lot lines to protect structures and property abutting the parking lot.

3.325.5 SURFACE AND DRAINAGE - All car parking areas shall be tar or bitumen sealed and/or graded with rolled or compacted cinders, gravel or other approved non-absorbent materials to prevent raising of dust and shall be maintained to prevent drainage onto adjoining properties or any footpaths adjoining the park.

3.325.6 ELECTRIC LIGHTING OF CAR PARKS - Wiring shall be provided either underground or on approved standards with shielded or semi-protected lighting units to furnish adequate illumination of driveways and lanes required by the Local Authority having jurisdiction for street lighting, but such illumination shall not be less than two and three-quarters lux (2.75 lx) over the whole of the parking area.

3.326 Mobile Units

3.326.1 DEFINITION OF - A structure of vehicular, portable design built on a chassis and designed to be moved from one site to another, and to be used with or without a permanent foundation, usually for residential, office, or similar uses.

NOTE: Before issuing a permit for a mobile unit, often referred to as "Mobile Homes" in some countries, Local Authority officers will havé to satisfy themselves that the unit is not, in fact, going to be used for a long period in one location. If such is the case, then that section of the Code dealing with pre-fabricated buildings (Part 5 Section 2 of this Code) must be complied with.

3.326.2 GENERAL - Mobile units as defined above shall be designed, constructed and maintained to be transported from one location to another and not mounted on a permanent foundation. A mobile unit placed on a permanent foundation or on foundation piers shall be designed and constructed to comply with all of the requirements of this Code for in situ and pre-fabricated construction. Most Local Authorities have by-laws covering the periods for which mobile units/mobile homes are permitted to remain on one site or section and these by-laws must be strictly adhered to.

3.326.3 CONSTRUCTION - Residential mobile units shall be of an approved design and construction in accordance with the applicable ordinances and statutes. All other mobile units shall be designed and constructed in accordance with the requirements of this Code. All mobile units on a permanent foundation shall be evaluated, inspected and labelled in accordance with Part 5 Section 2 of this Code.

3.326.4 LOCATION - Mobile units shall be located in spaces approved for such use. The provision of this Code shall not be construed to repeal, modify or constitute an alternative to any lawful zoning regulations. In case of conflict between this Code or any other ordinance or statute, the most rigid requirements shall apply.

3.326.5 ANCHORING/TIEING DOWN OF UNITS - Every parking space for mobile units shall be provided with devices for anchoring the unit to prevent overturning or uplift. The owner of the parking space shall anchor or cause to be anchored all mobile units located on the parking space. Where concrete platforms are provided for the parking of the units anchorage may be by eyelets imbedded in the concrete with adequate anchor plates or hooks; or other suitable means. The anchorage shall be adequate to withstand wind forces and uplift as required in Part 2 of this Code, the section dealing with Structural and Foundation Loads and Stresses for buildings and structures, based upon the size and weight of the units.

3.327 **Motels**

3.327.1 GENERAL - All buildings and accessory structures erected for or used as motels shall comply with the requirements and limitations of this Code for the occupancy and use for which they are designed and as herein specifically required.

3.327.2 GARAGES - Garages when attached to motel residential buildings shall have the interior faces of all walls, when not of approved masonry construction, and the ceiling protected to afford one (1) hour fire-resistance rating and all connecting openings shall be protected with approved three-quarter (3/4) hour fire doors or their equivalent complying with Part 3 Section 6 of this Code or with forty-five millimetres (45 mm) solid core wood doors. Roofed-over passageways may be used to connect garages to dwellings if protected with one (1) hour fire-resistance rated construction.

3.327.3 REQUIRED EXITWAYS - All exitways in buildings more than one (1) storey in height shall be constructed of one (1) hour fire-resistance rating and all stories above the ground floor shall have at least two (2) means of egress complying with Section 3 Part 5 of this Code. All exitways from residential quarters shall lead to open spaces not less than six metres (6.0 m) in width which provide direct access to public streets or highways.

3.327.4 DRIVEWAYS, ACCESS WAYS AND PARKING SPACES - The arrangement and capacity of driveways, lanes and parking spaces shall comply with the requirements specified for parking lots in clauses under 3.325 above.

3.327.5 WATER SUPPLY AND SANITARY FACILITIES - Fresh water supply for drinking and domestic purposes and all sanitary facilities shall comply with the provisions of the Plumbing Code, in Part 4 Section 4 below.

3.328 **Radio and Television Towers**

3.328.1 GENERAL - Subject to the structural provisions of Part 2 Section 2 for wind loads and the requirements of Part 3 Section 6 (3.625) governing the fire-resistance ratings of buildings for the support of roof structures, all radio and television towers shall be designed and constructed as herein provided.

3.328.2 LOCATION AND ACCESS - The towers shall be so located and equipped with step bolts and ladders to be readily accessible for inspection purposes. Guy wires or other accessories shall not cross or encroach upon any street or other public spaces, or over any electric power lines, or encroach upon any other privately owned property without written consent of the owner.

3.328.3 CONSTRUCTION - All towers shall be constructed or approved corrosion-resistive non-combustible materials. Within the limitations of Part 3 Section 2 (3.203) for fire limits, isolated radio towers may be constructed of timber sizes qualifying for mill type construction when not more than thirty metres (30.0 m) in height.

3.328.4 LOADS - The structures shall be securely braced and anchored to resist a wind of not less than one hundred and fifty kilograms (150kg) per square metre ($1.0\ m^2$) i.e. one and one-half kilopascals (1.5 kPa) on the net area of both sides of latticed construction and on the projected area of the antennae. Where subject to winds of unusual velocity, the loads shall be increased accordingly. Due allowance shall be made for effect of shape or individual elements and contour of the tower as provided in Part 2 Section 2 of this Code.

3.328.5 DEAD LOAD - Antennae and towers shall be designed for the full dead load as per Part 2 Section 1 of this Code.

3.328.6 UPLIFT - Adequate foundations and anchorage shall be provided to resist twice (2 times) the calculated wind uplift.

3.328.7 GROUND/EARTHING - All towers shall be permanently and effectively earthed/grounded in an approved and acceptable manner (see also Part 4 Section 2 of this Code).

3.329 **Radio and Television Antennae:**

3.329.1 WHEN PERMITS NOT REQUIRED - Antennae structures for private radio or television reception not more than five metres (5.0 m) in height may be erected and maintained on the roof of any building without a building permit. Such a structure, however, shall not be erected so as to injure the roof covering and when removed from the roof, the roof covering shall be repaired to maintain weather and water tightness. The installation shall not be erected nearer to the boundary line than the total height of the antennae structure, nor shall such structure be installed near electric power lines or encroach upon any street or other public space.

3.329.2 WHEN PERMITS ARE REQUIRED - The approval of the building authority shall be secured for all antennae structures more than five metres (5.0 m) in height. The application shall be accompanied by detailed drawings of the structure and methods of anchorage. All connections to the roof structure must be properly flashed to maintain water tightness. The design and materials of construction shall comply with the requirements of clause 3.328 above for character, quality, and minimum dimensions.

3.330 Swimming Pools

3.330.1 GENERAL - Pools used for swimming or bathing shall be in conformity with the requirements of this sub-section, provided, however, these regulations shall not be applicable to any such pool less than twenty-four square metres (24 m^2), except when such pools are permanently equipped with a water-recirculated system or involve structural materials. For purposes of this Code, pools are classified as private swimming pools or public and semi-public swimming pools, as defined in clause 3.331.2 below. Materials and construction used in swimming pools shall comply with the applicable requirements of this Code. Pools used for swimming or bathing and their equipment or accessories which are constructed, installed and maintained in accordance with the applicable standards shall be deemed to conform to the requirements of this Code, provided the requirements of clauses under 3.330 below are included in the installation.

3.330.2 CLASSIFICATION OF POOLS - Any constructed pool which is used or intended to be used, as a swimming pool in connection with a single-family residence and available only to the family of the householder and his private guests shall be classified as a private swimming pool. Any swimming pool other than a private swimming pool shall be classified as a public or semi-public swimming pool.

3.330.3 APPROVALS (i) PERMIT - A swimming pool or appurtenances thereto shall not be constructed, installed, enlarged or altered until a permit has been obtained from the Building Authority. The approval of all city, and/or country or other authorities having jurisdiction over swimming pools shall be obtained before applying to the building authority for a permit. Certified copies of these approvals shall be filed as part of the supporting data for the application for the permit.

3.330.4 APPROVALS (ii) PLANS - Plans shall accurately show dimensions and construction of pool and appurtenances and properly established distances to boundary line, buildings, walks and fences; details of water-supply system, drainage and water-disposal systems, and all appurtenances pertaining to the swimming pool. Detail plans of structures, vertical elevations, and sections through the pool showing depths shall be included.

3.330.5 LOCATIONS - Private swimming pools shall not encroach on any front or side yard required by this Code, or the governing zoning law, except by specific rules of the jurisdiction in which it may be located. A wall of a swimming pool shall not be located less than two metres from rear or side property line, except by specific rules of the jurisdiction in which it may be located or by a special dispensation from such jurisdiction or Local Controlling Authority.

3.330.6 STRUCTURAL DESIGN - The pool structure shall be engineered and designed to withstand the expected forces to which it will be subjected.

3.330.7 WALL SLOPES - To a depth up to one and one-half metres (1.5 m) from the top, the wall slope shall not be more than sixty centimetres (0.6 m) horizontal in one and one-half metres (1.5 m) vertical.

3.330.8 FLOOR SLOPES - The slope of the floor on the shallow side of transition point shall not exceed one (1) vertical to seven (7) horizontal. The transition point between shallow and deep water shall not be more than one and one-half metres (1.5 m) deep.

3.330.9 SURFACE CLEANING - All swimming pools shall be provided with a recirculating skimming device or overflow gutters to remove scum and foreign matter from the surface of the water. Where skimmers are used there shall be at least one (1) SKIMMING DEVICE FOR EACH one hundred square metres of surface area or fraction thereof. Where overflow gutters are used they shall be not less than seventy-five millimetres (75 mm) deep with falls of not less than one in fifty (1 in 50) to drains, and constructed so they are safe, cleanable and that matter entering the gutters will not be washed out by a sudden surge of entering water.

3.330.10 WALKWAYS - All public or semi-public swimming pools shall have walkways not less than one and one-quarter metres (1.25 m) in width extending entirely around the pool. Where curbs or sidewalks are used around any swimming pool they shall have a non-slip surface for a width of not less than three hundred millimetres (300 mm) at the edge of the pool and shall be so arranged to prevent return of surface water to the pool.

3.330.11 STEPS AND LADDERS - One (1) or more means of egress shall be provided from the pool. Treads of steps or ladders shall have non-slip surfaces and handrails on both sides, except that handrails may be omitted when there are not more than four (4) steps or when they extend the full width of the side or end of the pool.

3.330.12 WATER SUPPLY AND TREATMENT - All swimming pools shall be provided with a potable water supply, free of cross-connections with the pool or its equipment. All public and semi-public swimming pools shall have equipment designed and installed so that there is a pool water turnover at least once every eight (8) hours. Filters shall not filter water at a rate in excess of one hundred and twenty-five litres (125 l) per minute per square metre (m^2) of surface area. The treatment system shall be so designed and installed to provide in the water, at all times when the pool is in use, excess chlorine of not less than four-tenths of one part per million (0.4 ppm) or more than six-tenths (0.6 ppm) or excess chloramine between seven-tenths (0.7) and one part per million (1.0 ppm), or disinfectant may be provided by other approved means. Acidity/Alkalinity of the pool water shall not be below seven (7.0) or more than seven and one-half (7.5). All recirculation systems shall be provided with an approved hair and lint strainer installed in the system ahead of the pump. Private swimming pools shall be designed and installed so that there is a pool water turnover at least once every eighteen (18) hours. Filters shall not filter water at a rate in excess of two hundred litres per minute (200 l) per square metre (1.0 m^2) of surface area. The pool owner shall be instructed in proper care and maintenance of the pool, by the supplier or builder, including the use of high test calcium hypochlorite (dry chlorine) or sodium hypochlorite (liquid chlorine) or equally effective germicide and algaecide and the importance of proper pH (alkalinity and acidity) control.

3.330.13 DRAINAGE SYSTEMS - The swimming pool and all of its pumping and filtering equipment shall be so designed and installed so that it can be completely emptied of water and the discharged water shall be disposed of in an approved manner that will not create a nuisance to adjoining property, or streets adjacent.

3.330.14 SWIMMING POOL APPURTENANT STRUCTURES - All appurtenant structures, installations, and equipment, such as showers, dressing rooms, equipment houses or other buildings and structures, including plumbing, heating, and air conditioning, amongst others appurtenant to a swimming pool, shall comply with all applicable requirements of this Code and the zoning laws of the local controlling authorities.

3.330.15 ACCESSORIES - All swimming pool accessories shall be designed, constructed, and installed so as not to be a safety hazard. Installations or structures for diving purposes shall be properly anchored to insure stability, and properly designed and located for maximum safety.

3.330.16 SAFETY PRECAUTIONS (i) EQUIPMENT INSTALLATIONS - Pumps, filters, and other mechanical and electrical equipment for public and semi-public swimming pools shall be enclosed in such a manner so as to be accessible only to authorized persons and not to bathers. Construction and drainage shall be such as to avoid the entrance and accumulation of water in the vicinity of electrical equipment.

3.330.17 SAFETY PRECAUTIONS (ii) SWIMMING POOL - Every person owning land on which there is situated a swimming pool, which contains six-tenths of one metre (0.6 m) or more of water in depth at any point, shall erect and maintain thereon an adequate enclosure either surrounding the property or pool area, sufficient to make such body of water inaccessible to small children. Such enclosure, including gates therein, must be not less than one and one-quarter metres (1.25 m) above the surrounding ground; all gates must be self-latching with latches placed one and one-quarter metres (1.25 m) above the ground or otherwise made inaccessible from the outside to small children. A natural barrier, hedge, pool cover or other protective device approved by the governing body may be used so long as the degree of protection afforded by the substituted devices or structures is not less than the protection afforded by the enclosure, gate and latch described herein.

3.331 Open Parking Buildings (for Cars)

3.331.1 GENERAL - Open passenger vehicle parking structures are those structures used for the parking and/or storage of passenger motor vehicles designed to carry not more than nine (9) persons, and include the following two (2) general types.

(i) Ramp-type parking structures are those employing a series of continuously rising floors or a series of interconnecting ramps between floors permitting the movement of passenger automobiles under their own power to and from the street level; and

(ii) Mechanical-type parking structures are those employing specially designed parking machines, elevators, lifts, conveyors, moving cranes, dollies or other devices for moving passenger automobiles to and from the street level.

For exitway requirements see Means of Egress Part 3 Section 5 of this Code, clause 3.511.7.

3.331.2 GENERAL CONSTRUCTION REQUIREMENTS - Passenger vehicle structures shall be constructed of non-combustible materials throughout, including structural framing, floors, roofs and walls. Any enclosed rooms or spaces on the premises shall comply with the applicable requirements of this Code.

3.331.3 SEPARATIONS - Parking structures may be erected without exterior walls except that an enclosure wall with not less than two (2) hours fire-resistance rating, without openings therein, shall be provided when located within two metres (2.0 m) of interior boundaries unless such boundaries are adjacent to permanent open spaces (parks) right-of-way, or similar.

3.331.4 BASEMENTS - Basements, if used for parking of vehicles, shall be sprinklered in accordance with the provision of Part 3 Section 7 of this Code (3.703).

NOTE: (i) This clause, 3.331.4, refers specifically to this sub-section Open Parking Buildings (for cars) only, and does not necessarily embrace office or other buildings with basement car parks for its tenants.

(ii) Also there can be exceptions to this clause, 3.331.4, in certain cases: e.g. if the "open parking building" is only open on one side, usually a street side, and the basement is completely enclosed on all four sides with no openings to upper floors and the only openings are for access or egress of vehicles and these are not below the open side of upper floors, it is not required that such basement be sprinklered. The ruling of both the Local Authority and the Fire Authority must be sought on any such dispensations and their unanimous decison shall be strictly adhered to.

3.331.5 PETROL/GAS DISPENSING - Areas used for dispensing of any motor fuels in parking buildings shall be located on the ground floors only, and they shall comply with the provisions of Part 3 Section 3, clauses under 3.316 above.

3.331.6 HEIGHTS AND AREAS FOR OPEN PARKING STRUCTURES -The heights and areas of open parking buildings shall not exceed the limits specified in the following Table 3.331.

3.331.7 PROTECTIVE GUARD RAILS - All wells, shafts and other open, exposed spaces throughout, except on ground floors where there are no basements or other floor openings, shall be enclosed and protected with continuous walls or protective guard rails at least one metre (1.0 m) in height, except that in those structures wherein vehicles are hoisted to the desired level and placed in the parking space entirely by approved mechanical means, the one metre (1.0m) high continuous wall or protective guard rail may be omitted on the side of the parking levels adjacent to the space occupied by the hoisting and placing equipment.

TABLE 3.331
HEIGHT AND AREA LIMITATION FOR OPEN PARKING STRUCTURES

Type of Construction	Height-Stories/Metres	Area in Sq. Metres
1-A and 1-B	Unlimited	Unlimited
2-A	12 stories - 35 m	Unlimited
2-B	10 stories - 30 m	15,000 m^2
2-C	8 stories - 25 m	9,000 m^2
2-B and 2-C (Note 1)	2 stories - 7.5 m	Unlimited

NOTE: 1:

(a) Type 2-B and 2-C construction may be up to six (6) stories in height and unlimited in area when at least 50 percent (50%) open on all sides and when the horizontal distance from any point on any level to an exterior wall opening on a street, alley court yard or any other open space exceeds thirty metres (30.0 m).

(b) Type 2-B and 2-C construction when fully sprinklered may be of unlimited area.

(c) All above limits of height permit car parking on the roof.

3.331.8 WHEEL GUARDS - Wheel guards made of non-combustible material shall be placed wherever required.

3.332 Fall-Out Shelters

3.332.1 GENERAL - This section of the Code shall establish the minimum criteria which must be met before a building or building space can be constructed, occupied, used, or designated as a fall-out shelter, and such shelters must be constructed in accordance with applicable standards such as Concrete Construction, Fire Protection, and Safety Practices, also up-to-date information may be obtained through the U.S. Department of Defence publications obtainable from the Office of the Secretary of the Army, Washington, D.C. 20390, USA or British Defence Manual, Ministry of Defence, London, England.

3.333 **High Rise Buildings**

3.333.1 APPLICABILITY - The provisions of this section shall apply to all buildings of the following occupancy groups when such buildings have floors used for human occupancy located more than seven (7) stories or twenty-five metres (25 m) above the lowest level of fire department vehicle access:

(i) Occupancy group B (Business)
(ii) Occupancy group R-1 (residential hotel) and
(iii) Occupancy group R-2 (residential multi-family)

3.333.2 MAINTENANCE AND INSPECTION - All fire protection systems shall be maintained in an operative condition at all times and shall be periodically inspected and tested in accordance with the Fire Prevention Code. Maintenance inspections shall be made quarterly and logged in a journal kept available for inspection.

3.333.3 OPTIONS - All buildings shall be provided with either an approved automatic fire suppression system or safe areas of refuge (compartmentation) in accordance with the following.

3.333.4 AUTOMATIC FIRE SUPPRESSION SYSTEM - When provided as required herein, the automatic fire suppression system shall be installed throughout the building. The system shall be designed using the parameters set forth in Part 3 Section 7 of this Code and the following:

(i) Shut-off valves, and a water flow device shall be provided for each floor of the building.

(ii) In the two higher seismic zones (U.S. 2 and 3 - other 1 and 2) each system shall be supplied by two or more risers (rising mains). An approved check valve shall be provided and installed at each point of connection of the sprinkler system to the riser in such a manner that one of the interconnected risers can remain operational should a break occur in the other riser/main.

(iii) In addition to the main water supply in the two higher seismic zones, a secondary on-site supply of water equal to the hydraulically calculated sprinkler design demand plus three hundred and fifty litres (350 l) per minute additional for the total standpipe system shall be provided. This supply shall be automatically available if the principal supply fails and shall have a duration of thirty (30) minutes.

3.333.5 AUTOMATIC FIRE SUPPRESSION SYSTEM ALTERNATIVES - When a fire suppression system is installed modifications to this Code are permitted as described in the following.

(i) The type of construction required by this Code may be modified as follows:

Type of Construction set out in Table 3.114 (Part 3 Section 1)	Modified Type of Construction Permitted Hereunder
1-A	1-B
1-B	2-A
2-A	2-B

NOTE: For construction classification of buildings, refer to Part 3, Section 1 of this Code, Table 3.114.

(ii) The fire-resistance rating of exitway access corridors and vertical separation to tenant spaces shall not be required in occupancy group B (business) buildings; there shall be a minimum of one-half (1/2) hour in occupancy group R-1 (residential, hotel) and R-2 (residential, multi-family) buildings; and the wall or partitions may be terminated at the lowest portion of the fire-resistance rated assembly above.

(iii) Vertical shafts other than stairway enclosures and elevator hoistway enclosures may be reduced to one (1) hour when sprinklers are installed within the shafts at alternative floors.

(iv) The exitway access and common corridor doors need not meet the requirements of Part 3 Section 5 clauses 3.512.6 to 3.512.8 except they shall be self-closing and tight fitting.

(v) The exitway access travel distance set out in Table 3.509 (Part 3 Section 5, sub-section 3.509 onwards and Table 3.509 therein) may be increased to ninety metres (90 m).

(vi) Smoke-proof enclosures as set forth in section 3.519 (Part 3 Section 5 of this Code) may be omitted, but required stairways shall be pressurized to not less than thirty-five pascals (35 Pa) in the manner described in clause 3.519.10 (in Part 3 Section 5 of Code).

(vii) Spandrel walls, eyebrows and compartmentation are not required; however, the fire-resistance rating of the floors and junctures of exterior walls with each floor must be maintained.

(viii) Fire dampers, other than those needed to maintain the fire-resistance rating of the floor-ceiling assembly, are not required. Where fire dampers will interfere with the operation of the smoke control system approved alternative protective devices shall be utilized.

(ix) Openable windows required by clauses 3.511.4 (Part 3 Section 5 of this Code) for emergency egress or rescue may be omitted.

3.333.6 AUTOMATIC FIRE SUPPRESSION ALTERNATIVES (B) - AREAS OF REFUGE - As an alternative to automatic fire suppression systems, areas of refuge conforming to the following may be provided:

(i) Every story shall be divided into two (2) or more areas of approximately the same size with no single area exceeding fourteen hundred square metres (1400 m^2). The wall and doors between the areas of refuge shall be constructed as required for a horizontal exit in sub-section 3.515 and onwards (Part 3 Section 5 of this Code).

(ii) Each area of refuge (compartment) shall contain a minimum of one (1) enclosed exitway stairway and each compartment shall have access to an elevator which may serve additional compartments. When elevators are directly accessible to more than one (1) compartment, the elevator lobby shall be separated from the compartments by not less than two (2) hour fire-resistance rated construction with tight-fitting opening protectives having fire-resistance ratings of not less than one and one-half (1/2) hour. (Elevators are not to be used as a means of egress).

(iii) Openings in exterior walls, where such openings are within one and one-half metres (1.5 m) of each other horizontally on adjacent floors or located vertically above one another, shall be protected by approved flame barriers either extending three-quarters of one metre (0.75 m) beyond the exterior wall in the plane of the floor or by approved vertical panels complying with clause 3.608.3 (Part 3 Section 6 of this Code).

(iv) Walls used for compartmenting a building shall have a fire-resistance rating of not less than two (2) hours. Duct penetrations of this wall shall not be permitted. Ferrous or copper piping and conduit may penetrate or pass through the wall only if the openings around such piping and conduit are sealed with impervious non-combustible materials sufficiently tight to prevent the transfer of smoke or combustion gases from one (1) side of the wall to the other and are so maintained. The fire door serving as the horizontal exit between compartments shall be so installed, fitted and gasketed that it will provide a substantial barrier to the passage of smoke and shall comply with clause 3.515.3 (Part 3 Section 5 of this Code).

(v) The fire-resistive floor or the floor/ceiling construction shall extend to and be tight against the exterior wall so that the fire-resistive integrity between stories is maintained. No penetrations or other installations which will impair the fire-resistive integrity of the floor or floor/ceiling assembly shall be permitted (see clause 3.605.1 (Part 3 Section 6) of this Code).

(vi) A manual fire alarm system (pull boxes) shall be provided.

3.333.7 SMOKE DETECTION SYSTEMS - An approved smoke detector suitable for the intended use shall be installed in the following:

(i) Every mechanical equipment room, electrical, transformer, and telephone equipment room, elevator machine room, or any similar rooms unless such rooms are protected with an automatic fire suppression system, and

(ii) Each connection to a vertical duct or riser serving two (2) or more stories from return air ducts or plenums of heating, ventilating and/or air-conditioning systems, except that in occupancy group R (residential) occupancies, an approved smoke detector may be used in each return air riser carrying not more than one hundred and fifty cubic metres (150 m^3) per minute and serving not more than ten (10) air inlet openings.

The actuation of any detector required by this section shall operate the alarm system and shall place into operation all equipment necessary to prevent the recirculation of smoke.

3.333.8 ALARM AND COMMUNICATING SYSTEMS - Alarm and communications systems shall be provided. These systems shall be so designed and installed that damage to any terminal unit will not render more than one section or zone of the whole system inoperative. A single communication system may be designed to serve a voice alarm, a public address system, and fire department communication system.

3.333.9 SEISMIC REQUIREMENTS - (referred to also under 3.333.4 (ii) above). In the numerous "earthquake prone" countries of the world there are three methods of determining heavy, medium, and light seismic zones thus:

	Heavy*	Medium*	Light	
(i)	3	2	1	USA
(ii)	A	B	C	Japan/New Zealand
(iii)	1	2	3	other

* For purposes of clarification in this Code, reference is made to these as "Heavy Zone", and medium zones rather than referring to seismic zones 1 and 2 or zones 2 and 3 or A and B.

In all high rise buildings erected in areas designated as heavy and medium seismic/earthquake zones the anchorage of the following mechanical and electrical equipment shall be designed in accordance with Part 2 Section 3 of this Code for a lateral force based on Cp and/or G as required by that section unless data approved by the building authority substantiating a lesser value is furnished:

(i) elevator drive and suspension systems;
(ii) emergency power and lighting facilities,
(iii) fire pumps and all other fire protection equipment and systems.

3.334 Covered Malls

3.334.1 SCOPE - All covered mall buildings are subject to the special requirements of this sub-section and can be divided into two types, A and B. Type A covered mall buildings are subject to the general provisions of this Code, while type B covered mall buildings may be designed and constructed in accordance with the special provisions as noted herein. All other applicable provisions not specifically specified herein shall be complied with.

3.334.2 REQUIREMENTS - For type B covered mall buildings, the following requirements as set out in 3.334.4 to and including 3.334.24 shall apply.

3.334.3 LEASE PLAN - The permit holder shall provide both the Building and Fire Departments with a lease plan showing the locations of each occupancy and its means of egress after the certificate of occupancy has been issued. Such plans shall be kept current. No modifications or changes in occupancy or use shall be made from that shown on the lease plan without prior approval of the Building Authority.

3.334.4 TENANT SEPARATIONS - Each tenant shall be separated from adjoining tenants by a wall having a minimum one (1) hour fire-resistance rating which shall extend from the floor to the underside of the ceiling. No separation is required between a tenant space and mall.

3.334.5 EXITWAYS - Exitways shall be provided in accordance with the following: 3.334.6 to and including 3.334.14.

3.334.6 The maximum length of exitway access travel from any point within the mall to an approved exitway along the natural and unobstructed path of travel shall not exceed fifty metres (50.0 m).

3.334.7 Each individual occupancy within the covered mall building shall be provided with a means of egress in accordance with other provisions of this Code. Measurements may be made to the entrance of the mall.

3.334.8 When the length of travel from the most remote point within a tenant space exceeds thirty metres (30 m) to the mall, a second means of egress shall be provided. When two (2) or more means of egress are required, the secondary exits may open into the mall, an exit corridor, an exit enclosure, or to the exterior. When a corridor provides the second means of egress, it shall be of one (1) hour fire-resistance rated construction and doors to the corridors shall be of one (1) hour opening protectives. Such doors shall be self-closing, and be so maintained, or shall be automatic closing when actuated by smoke detectors.

3.334.9 Large retail stores and supermarkets within the mall general area shall provide the required number of exitways and units of exit width directly to the exterior/open air. The occupant load of such stores (sometimes called "anchor stores") opening into the mall shall not be included in determining exitway requirements for the mall.

3.334.10 The dead end of any mall shall not exceed twice its width.

3.334.11 In determining required exitway facilities of the mall, the number of occupants for whom exitway facilities are to be provided, shall be based on the gross leasable area of the covered mall building (including the large (anchor) stores) and shall be based on the following:

Square Metres per person	Gross leasable area in m^2
2.75	under 27,500
3.75	27,500 to 65,000
4.5	over 65,000

3.334.12 The minimum width of exitway access passageways and corridors from a mall shall be one and one-half metres (1.5 m).

3.334.13 The required units of exit width and exitways shall be distributed equally through the mall.

3.334.14 Storage is prohibited in exitway corridors which are also used for service to the tenants. Such corridors shall be posted with conspicuous signs so stating.

3.334.15 MALL WIDTH - The minimum width of the mall shall be six metres (6.0 m) and there shall be a minimum of three metres (3.0 m) clear exitway width to a height of two and four-tenths metres (2.4 m) between any projection of a tenant space bordering the mall and the nearest kiosk, vending machine, bench, display opening, or other obstruction to egress travel.

The mall width shall be sufficient to accommodate the occupancy load emptying into the immediately adjacent mall as determined by clauses 3.334.5 to 3.334.14 above for all occupancies except assembly which shall be determined by sub-section 3.508 (Part 3 Section 5 of this Code).

3.334.16 TYPE OF CONSTRUCTION -

(i) The structural elements of the covered mall building shall be non-combustible (Type 1 and 2) or heavy timber (Type 3-A) construction.

(ii) Floor/ceiling assemblies and their supporting columns and beams within multi-level covered malls shall be of one (1) hour fire-resistance rated non-combustible construction.

(iii) Separation between tenant spaces and the mall is not required. When walls are provided, they shall comply with the provisions of Table 3.114 (Part 3 Section 1 of this Code) for other non-bearing partitions.

3.334.17 ROOF COVERINGS - Roofing for covered mall buildings shall be class A, B, or C as required by sub-section 3.626 (Part 3 Section 6 of this Code.)

3.334.18 MIXED OCCUPANCY - (i) Occupancy groups assembly (A), business (B), mercantile (M), and residential (R) may be accessory to the covered mall building. Accessory occupancies may be three (3) times the area permitted by Table 3.206 in Part 3 Section 2 of this Code.

Exception: Assembly (A) occupancies shall be so located in the covered mall building, that their main entrance is immediately adjacent to a principal entrance to the mall.

3.334.19 MIXED OCCUPANCY CLARIFICATION - (ii) It should be noted that the sprinkler system required in covered mall buildings (see 3.334.20 below) shall not be substituted for the required one hour fire-resistance rated construction. Assembly (A) occupancies other than restaurants shall have not less than one-half (1/2) of their required exitways opening directly to the exterior of the covered mall building.

3.334.20 FIRE PROTECTION - Every covered mall building shall be provided with fire protection equipment as follows:

(i) The covered mall and all buildings connected thereto shall be provided throughout with an approved fire suppression system. The suppression system in the covered mall shall be independent of the suppression systems in the buildings connected to the covered mall.

(ii) All sprinkler control valves shall be electrically supervised and connected to either the fire department or to an approved supervisory service.

(iii) Fire department standpipe outlets shall be provided within the mall at each entrance to an exit passageway, corridor or enclosed stairway and at exterior exits.

(iv) First-aid fire extinguishers shall be provided as required by the fire prevention Code.

3.334.21 FIRE EMERGENCY VENTILATING SYSTEM - The covered mall and exitway corridors serving the mall shall be equipped with an approved automatic exhaust system capable of producing six (6) air changes per hour computed on volume measured to a height of three and one-half metres (3.5 m) above each pedestrian area. Necessary outside air to accomplish the six (6) air changes per hour shall be provided. The exhaust system shall be activated by smoke detectors complying with the applicable standards, by operation of the sprinkler system, and manually. The activation system shall be installed in an approved manner. Exhaust shall be taken uniformly from the entire mall area and exitways serving the mall through an approved duct system with vents spaced not more than fifteen metres (15m) or through a ceiling plenum with uniformly distributed openings. Where tenant spaces are open to the mall area, exhaust may be taken through the tenant spaces. The approved automatic exhaust system may be a separate system or may be integrated with an approved air-conditioning system. Where a separate system is provided, operation of the fire emergency ventilating system shall automatically shut down the air-conditioning system or any other devices which interfere with the effective operation of the fire emergency ventilating system.

Exception:

(i) When mall buildings are of (generally) single storey construction; and

(ii) When the roof of the actual mall/public area is not less than one and one-half metres (1.5 m) above the roof(s) of shops opening off mall; and

(iii) Provided that the area of wall above shop roofs and below actual mall roof has opening windows, louvres of other approved vents in it of not less than fifty percent (50%) of its area, then the above clause 3.324.21 may, with the approval of the Local Authority, be disregarded.

3.334.22 FIRE DEPARTMENT ACCESS TO EQUIPMENT - Controls for air-conditioning systems, sprinkler risers and valves, or other fire detection, suppression or control elements shall be accessible to and properly identified with approved signs for use by the Fire Department.

3.334.23 PLASTIC PANELS AND PLASTIC SIGNS - Within every storey or level and from side wall to side wall of each tenant, approved plastic panels and signs shall be limited as follows:

(i) They shall not exceed twenty (20) percent of the wall area facing the mall.

(ii) They shall not exceed a height of one metre (1.0 m), except if the sign is vertical, the height shall not exceed two and one-half metres (2.5 m) and the width shall not exceed one metre (1.0 m).

(iii) They shall be located a minimum distance of one-half of one metre (0.5 m) from adjacent tenants.

(iv) All edges and the backs shall be fully encased in metal.

3.334.24 KIOSKS - Kiosks and similar structures (temporary or permanent) shall meet the following requirements:

(i) Combustible kiosks or other structures shall not be located within the covered mall unless constructed of fire-retardant-treated wood throughout, conforming to the relevant standards.

(ii) Kiosks or similar structures located within the covered mall shall be provided with approved fire-suppression and detection devices.

(iii) The minimum horizontal separation between kiosks and other structures within the covered mall shall be six metres (6.0 m).

(iv) Kiosks or similar structures shall have a maximum area of thirty square metres (30 m^2).

Exceptions:

(i) Kiosks that do not exceed ten square metres (10 m^2) in area and are permanently and solely for the sale of non-combustibles such as crockery, pottery, hardware and the like in malls that are fully protected, with a fire suppression system, need not, with the prior approval of the Local Authority, comply with 3.334.24 (ii), but must comply with all other requirements.

(ii) Temporary kiosks or "stands" erected for special uses such as sale of theatre or lottery tickets, or for some special fete advertising or similar uses and provided they are temporary and will be demolished within thirty days (30 days), with the prior approval of the Local Authority, need not comply with 3.334.24.

CARIBBEAN UNIFORM BUILDING CODE

PART 3

OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 4

LIGHT, VENTILATION AND SOUND TRANSMISSION CONTROLS

PART 3

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PART 3

SECTION 4

3.400 LIGHT, VENTILATION AND SOUND TRANSMISSION CONTROL

3.401 **Scope**

The provision of this section shall govern the means of light and ventilation required in all habitable and occupiable spaces and rooms. Every building and structure hereafter erected and every building room or space which is changed in use shall be constructed, arranged and equipped to conform to the requirements of this section and applicable to other relevant standards.

3.401.1 CONFLICTING LAWS OR REGULATIONS - The provisions covered in this section of the Code shall not be construed to nullify the provisions of any other law, ordinance, regulation, or by-law regulating yards, courts, light areas or other spaces required for light, ventilation or fire egress; but the greater requirements shall control the construction, and the decision of the local body shall be regarded as final and binding.

3.401.2 BUILDINGS ON SAME SITE/LOT/BUILDING SECTION - If more than one building is hereafter allowed to be placed on a single building site, or if a building is placed on the same lot with existing buildings, the several buildings may be treated as a single structure for the purpose of this article, provided equivalent uncovered lot area or other adequate sources of light and ventilation are furnished for all habitable and occupiable spaces and rooms.

3.402 **Plans and Specifications**

3.402.1 GENERAL - Plans for all buildings other than one and two-family and multi-family dwellings, which are designed for human occupancy, shall designate the number of occupants to be accommodated in various rooms and spaces, and when means of artificial lighting, air-conditioning, and/or ventilation are required, the application shall include sufficient details and description of the mechanical system to be installed as herein required or as specified in applicable British, American, or Canadian Standards, and also as required by other sections and parts of this Code, e.g. Part 3 Section 3 (General Building Limitations) and Part 4 (Services, Equipment and Systems).

3.403 **Standards of Natural Light**

3.403.1 GENERAL - In the application of the provisions of this section, the standard of natural light for all habitable and occupiable rooms, unless otherwise specifically required by the provisions of Part 3 Section 3 of this Code for special occupancies, shall be based on seventeen lux (17 lx) of illumination on the vertical plane adjacent to the exterior of the light-transmitting device in the enclosure wall and shall be adequate to provide an average illumination of four-tenths of one lux (0.4 lx) over the area of the room at a height of seven hundred and fifty millimetres (750 mm) above the floor level.

3.404 **Standards for Natural Ventilation**

3.404.1 GENERAL - In the application of the provisions of this section the standard of natural ventilation for all habitable and occupiable rooms shall be based on a volume of eleven and one-half cubic metres ($11.5m^3$) of air per occupant with ventilating skylights, monitors, louvres, windows, transoms, doors, or other alternative ventilating devices located in the exterior walls or on the roof of the building as provided in sub-sections 3.407 to 3.415 inclusive.

3.405 **Artificial Light and Ventilation**

3.405.1 WHEN REQUIRED - When natural light and ventilation do not meet the minimum requirements of this Code, or when rooms, which by use or occupancy, involve the presence of dust, fumes, gases, vapours or other noxious or deleterious impurities that create a fire or health hazard, or when required by the provisions of Part 3 Section 3 of this Code for special uses, the building shall be equipped with artificial light and mechanical means of ventilation under the conditions and of the minimum capacity prescribed herein and/or in the Mechanical Code Part 4 (Services Equipment and Systems).

3.405.2 OPERATION OF VENTILATING SYSTEMS - Where mechanical ventilation is accepted as an alternate for natural means of ventilation, or is required under the conditions herein prescribed, the system, equipment and distributing ducts shall be installed in accordance with the provisions of Part 4 Section 1 of this Code (Chimneys, flues, and vent pipes) and/or to comply with other relevant mechanical codes such as BSI, ASTM or Canadian. Ventilating systems shall be kept in operation at all times during normal occupancy of the building or space so used.

3.405.3 HABITABLE ROOMS - The glazed areas of windows and exterior doors in habitable rooms and spaces need not be openable where an approved mechanical ventilation system is provided capable of producing two changes of air per hour. Recirculation of not more than seventy-five percent of the air supplied may be permitted in habitable rooms except kitchens, provided the air recirculated does not come from a plenum or system fed with air returned from habitable rooms occupied by other families or from stairways or common hallways; except that recirculation of one hundred percent of the air supplied may be permitted if the system supplied only a single dwelling unit.

3.406 **Existing Buildings**

3.406.1 UNSAFE CONDITIONS - In all existing rooms or spaces in which the provisions for light and ventilation do not meet the requirements of this Section and which in the opinion of the Local Building Authority, are dangerous to the health and safety of the occupants, they shall order the required repairs or installations to render the building or structure livable for the posted use and occupancy load.

3.406.2 ALTERATIONS - A building shal not hereafter be altered or re-arranged so as to reduce either the size of a room, or the fresh air supply, or the amount of available natural light to less than that required for buildings hereafter erected; or to create an additional room unless made to conform to the requirements of sub-section 3.407. The Building Authority may permit new rooms to be of the same height as existing rooms in the same storey unless in its opinion greater provision of artificial light and ventilation is deemed necessary to insure healthful living conditions.

3.406.3 (UNCOVERED) YARDS AND COURTS - No buildings shall hereinafter be enlarged, nor shall the size of the section/lot on which it is located be reduced or diminished, so as to decrease the areas and/or dimensions of required courts or yards to less than that prescribed in this section, for the lighting and ventilation of new buildings.

3.406.4 EXCEPTIONS - The Local Authority may grant a dispensation in regards to 3.406.3 above where:

(i) an adjoining building has been demolished or reduced in height since the building under consideration for alterations was erected; or

(ii) when an adjoining site or lot has been converted to a street, road, park or right of way since the building under consideration for alterations was erected.

3.407 Natural Lighting and Ventilation of Rooms

3.407.1 WINDOWS AND SKYLIGHTS - All habitable and occupiable rooms or spaces shall contain windows, skylights, monitors, glazed doors, transoms, glass block panels, or other light-transmitting media opening to the sky or on a public street, yard or court complying with the provisions of this Section. The light-transmitting properties and the area of the devices used shall be adequate to meet the minimum daylighting and ventilating requirements specified herein and in the approved rules as covered below.

3.407.2 WINDOW AND DOOR SIZES - Windows and exterior doors may be used as a natural means of light and ventilation, and when so used their aggregate glass areas shall amount to not less than ten percent of the floor area served, and with not less than one-half of this required area available for unobstructed ventilation, that is, opening area of windows (or doors).

3.407.3 CROSS VENTILATION - Although not a definite requirement of this Code, it is very strongly recommended that when architects, planners and designers are complying with this Code, and in particular, these clauses 3.407.1 to 3.407.8 they endeavour to obtain cross ventilation wherever possible. Opening doors and/or windows in opposite walls are the best form of cross ventilation, while opening doors/ windows in adjoining walls are still better than all the ventilation being in one wall only.

3.407.4 OPENINGS ON YARDS AND COURTS - In order to be credited as a source of natural light or ventilation under the provisions of this Section, a window or any other approved device shall open directly onto a public street, alley or other open public space, or on a yard or court located on the same lot or plot complying with the requirements of sub-sections 3.417, 3.418 and 3.419.

3.407.5 ROOM DIMENSIONS (AND AREAS) - These areas and/or dimensions are in all cases to be taken as an absolute minimum, and when any doubt exists, or where they may appear to be ambiguous, the ruling of the local body (Inspectors) shall be sought, and their decision shall be final.

3.407.6 CEILING HEIGHTS - Habitable (space) rooms, other than kitchens, storage rooms and laundry rooms shall have a ceiling height of not less than two and four-tenths metres (2.4 m). Hallways, corridors, bathrooms, water closet rooms, and kitchens shall have a ceiling height of not less than two and two-tenths metres (2.2 m) measured to the lowest projection from the ceiling. If any room in a building has a sloping ceiling, the prescribed ceiling height for that room is required over two-thirds of the area thereof, and the lowest point of the finished ceiling above finished floor shall not be less than one and eight-tenths of one metre (1.8 m). No portion of such room e.g. alcoves, bay or oriel windows, or storage areas, with a ceiling height of less than 1.8 metres shall be included in any computation of the minimum area thereof.

3.407.7 FLOOR AREAS - Every dwelling unit shall have at least one room which shall have not less than fourteen square metres (14 m^2) of floor area. Other habitable rooms except kitchens shall have an area of not less than six and one-half square metres (6.5 m^2).

3.407.8 WIDTHS - No habitable room other than a kitchen shall be less than two and two-tenths metres (2.2 m) in any dimension.

3.408 Lighting and Venting of Special Spaces

3.408.1 ALCOVE ROOMS - When alcove rooms open without obstruction into adjoining rooms, the required window openings to the outer air shall be based on the combined floor area of room and alcove. An alcove space shall not be more than six square metres (6.0 m^2) in area and the opening to the adjoining room shall be not less than eighty percent of the superficial area of the dividing wall, unless provided with separate means of light and ventilation.

3.408.2 ATTIC SPACES - All attic spaces and spaces between roofs and top floor ceilings shall be ventilated by not less than two opposite windows, louvres, or vents with a total clear area of opening not less than one percent of the horizontally projected roof area (in gabled roofs). In hipped roof, lean-to, or flat roofs provisions must also be made for ventilation of the roof spaces through vented soffit linings, roof or ridge vents or any other method approved by the Local Authority.

3.408.3 CRAWL SPACES (UNDER FLOOR AREAS) - In all buildings constructed without basements in which the ground floor is suspended and does not bear directly on to the ground (in wooden floors supported on bearing walls and/or piles, or suspended reinforced concrete floors) a space shall be provided under the ground floor of not less than one-half of one metre (0.5 m) in depth; such space shall be vented with screened openings (doors or vents) having a clear area of not less than one-half of one percent (0.5%) of the enclosed building area. When hollow floating mat foundations in concrete are provided, the requirement for ventilation shall not apply.

3.409 **Basements and Cellars**

3.409.1 GENERAL - Except as may be otherwise specified for habitable or occupiable rooms or specifically provided in Section 3 Part 3 of this Code for special uses, the glass window area in basements and cellars, except crawl spaces as provided in sub-section 3.408.3 shall be not less than one-fiftieth of the floor area served, and provisions shall be made for fresh air supply prescribed for specific uses in sub-section 3.415 and the Mechanical Section of this Code (Part 4).

3.410 **Business and Work Rooms**

3.410.1 GENERAL - Offices, stores, mercantile and salesrooms, restaurants, markets, bakeries, hotel and restaurant kitchens, factories, workshops, machinery and boiler rooms shall be provided with the required windows specified in sub-section 3.407 for habitable and occupiable rooms, opening directly on a street or required yard or court; or such rooms shall be equipped with an approved system of mechanical ventilation complying with sub-section 3.405 and the Mechanical Section of this Code.

3.411 **Assembly Rooms (A Group Occupancies)**

3.411.1 GENERAL - In addition to the requirements of Part 3 Section 3 of this Code for special uses, the required windows or other approved devices for natural ventilation shall be distributed as equally as practicable on at least two sides of the room; and artificial lighting shall comply with the requirements of this article and Part 4 Section 2 of this Code.

3.412 **Rooms of or in Institutional Buildings (I Group Occupancies)**

3.412.1 GENERAL - In buildings of the institutional occupancy group, every habitable and occupiable room shall be provided with light and ventilation as herein provided, except that in buildings used for enforced detention of people (use group I-I) indirect openings to the street or court may be permitted through intermediate corridors or by other approved means of light and ventilation.

3.413 **Bath and Toilet Rooms**

3.413.1 GENERAL - Every bath and toilet room shall be lighted and ventilated by one of the methods prescribed in clauses 3.413.2 through to and including 3.413.6.

3.413.2 EXTERIOR WINDOWS - Windows opening to the outer air as provided in subsection 3.407 but not less than three-tenths of one square metre (0.3 m^2).

3.413.3 VENT SHAFT WINDOWS - Windows as provided in subsection 3.407 but not less than three-tenths of one square metre ($0.3\ m^2$) in area, opening on a vent shaft with a cross-sectional area of three-tenths of one square metre ($0.3 m^2$) for every one metre (1.0 m) in height but not less than eight-tenths of one square metre ($0.8\ m^2$) in area, open to the outer air at top or constructed with equivalent side louvre openings.

3.413.4 VENTS AND DUCTS - Individual vents or ducts, may be used provided they are constructed of approved non-combustible materials complying with Part 4 Section 1 of this Code and have a minimum cross-sectional area of five hundred square centimetres ($500\ cm^2$) or ($0.05\ m^2$) with an additional five hundred square centimetres ($500\ cm^2$) in area for each additional water closet or urinal stall above two in number. Such ducts shall be of adequate height and so located as to ensure a minimum supply of six-tenths of one cubic metre ($0.6\ m^3$) of fresh air per one square metre ($1.0\ m^2$) of room area.

3.413.5 SKYLIGHTS - A skylight of approved non-combustible construction complying with Part 3 Section 6 clauses 3.625.3, 3.625.4 and 3.625.5 of this Code and not less than three-tenths of one square metre ($0.3\ m^2$) in area with ventilating opening.

3.413.6 MECHANICAL VENTILATING SYSTEMS - Any system of mechanical or gravity ventilation capable of producing a change of air every twelve minutes in private bathrooms. Public bathroom mechanical ventilation systems shall comply with the Mechanical Code - Part 4.

3.413.7 RECIRCULATION - Recirculation of air supplied to toilet rooms, bathrooms and rest rooms shall not be permitted.

3.413.8 ARTIFICIAL LIGHTING - Illumination shall be provided in all toilet rooms to afford an average intensity of one-fifth of one lux (0.2 lx) measured at a level of seven hundred and fifty millimetres (750 mm) above the floor.

3.414 Stairways and Exitways in Residential and Institutional Buildings

3.414.1 WINDOWS - In all multi-family dwellings (occupancy group R-2) and in Institutional buildings for the care or treatment of people (occupancy group I-2) required interior stairways shall be provided with windows to the outer air having a glass area of not less than one square metre ($1\ m^2$) which opens on to a street, alley, yard, or court, or with the equivalent source of light for each storey through which the stairway passes; and such additional artificial lighting to provide the equivalent illumination at all times that the building is occupied - as specified in sub-section 3.524 (Part 3 Section 5 of this Code).

3.414.2 SKYLIGHTS - When the building is not more than three (3) stories in height, a ventilating skylight of the required area may be used in lieu of windows.

3.414.3 HALLWAYS - Hallways should have at least one window opening directly on to a street or on to a required yard or court in each storey, located so that light penetrates the full length of the hallway, with additional windows for each change of direction of the hallway; or the equivalent artificial lighting shall be provided. Every recess or return with a depth or length which exceeds twice the width of the hall, and every corridor separately shut off by a door shall be treated as a separate hall, in applying the provisions of this Section of the Code.

3.414.4 MECHANICAL VENTILATING SYSTEMS - All exitways and common corridors in multi-family dwellings (occupancy group R-2) and in institutional (occupancy group I) buildings shall be provided with not less than forty-five hundredths of one cubic metre (0.45 m^3) per minute of fresh air per one square metre (1 m^2) of floor area. Not more than seventy-five percent of the air supplied shall be recirculated.

3.414.5 BUSINESS AND ASSEMBLY BUILDINGS - All stairway enclosures shall conform to the requirements of Part 3 Sections 5 and 6 of this Code for construction, and shall have the means of artificial illumination to meet the requirements of this Section 4.

3.414.6 INTENSITY OF ILLUMINATION - In all required exitways, except in one and two-family dwellings, and wherever natural lighting is not available, artificial lighting shall be provided to furnish not less than one-fifth of one lux (0.2 lx) at the floor level of all required exitways.

3.415 Required Fresh Air Supply

3.415.1 GENERAL - Mechanical or gravity systems of ventilation shall provide the minimum air changes per hour specified in this Code (and the Mechanical Code). Recirculation of air supplied to kitchens, lavatories, toilet rooms, bathrooms, rest rooms, laboratories and garages shall not be permitted.

3.416 Ventilation of Shafts - Other than Elevators or Dumbwaiter Hoistways

3.416.1 GENERAL - All enclosed vertical shafts extending through more than two stories of every building, except elevator or dumbwaiter hoistways, shall be automatically vented to the outer air as herein required or as specified in Part 3 Section 6 sub-section 3.612 of this Code.

3.416.2 EXTENDING TO ROOF - Shaft enclosures extending to the roof shall be provided with a metal skylight constructed to comply with Part 3 Section 6 clause 3.625.3 of this Code or with windows of equivalent area or with other approved automatic means of removing hot air and gases.

3.416.3 THERMOSTATIC CONTROL - The automatic operation of fire shutters, skylights and other vent relief devices may be controlled by suitable links designed to operate at a fixed temperature of not more than seventy degrees Celsius or by electric or pneumatic operation under a rapid rise in temperature of eight to eleven degrees (8^{o} - 11^{o}) Celsius per minute or by other approved methods.

3.416.4 NOT EXTENDING TO ROOF - Shaft enclosures NOT extending to the roof shall be provided with gas and smoke relief vents or adequate mechanical means of ventilation in conformity to the provisons of Part 3 Section 6 sub-section 3.612.

3.417 Courts - Light Areas

3.417.1 GENERAL - All courts and/or light areas required to serve rooms for light and ventilation purposes shall comply with the requirements of this section.

3.417.2 MINIMUM WIDTHS - Every such court or area shall have a minimum width of one-quarter of one metre (0.25 m) for each one metre (1.0 m) of height or fraction thereof, but not less than one and one-half metres (1.5 m) for outer courts and twice these values for inner courts.

3.417.3 IRREGULAR COURT WIDTH - In the case of irregular or gore-shaped courts, the required minimum width of a court may be deemed to be the average width; nor shall the length of any court be more than twice its width.

3.417.4 AREA OF COURT/LIGHT AREA - The cross-sectional area of a required court shall be not less than one and one-half times the square of its width; nor shall the length of any court be more than twice its width.

3.417.5 ACCESS TO COURTS/LIGHT AREAS - A door or other means of access shall be provided at the bottom of every court that is not otherwise conveniently accessible for purposes of cleaning.

3.417.6 AIR INTAKES TO COURT LIGHT AREAS - Every court serving one or more habitable rooms that does not open for its full height on one or more sides to a street or legal yard shall be connected at or near the bottom with a street or yard by a horizontal intake or passage of fire-resistive construction. Such intake or passage shall have a cross-sectional area of not less than two square metres (2.0 m^2), and shall remain fully open at both ends and unobstructed for its full size and length, except that grilles of non-combustible construction complying with the approved rules may be permitted at the ends of the intake.

3.417.7 FIRE-RESISTANCE - The walls, floors and ceilings of such intakes or passages shall have a fire-resistance rating of not less than two hours in buildings of Types 1, 2 or 3 construction and not less than one hour in Type 4 construction.

3.417.8 COURT WALLS - When in the opinion of the Building Authority, windows facing on courts do not receive adequate direct light by reason of peculiar arrangement or orientation, they may require the walls to be constructed of light coloured masonry, or to be painted and maintained in a light colour to furnish additional reflected light.

3.417.9 DRAINAGE TO COURTS/LIGHT AREAS - The bottom of every court shall be properly graded and drained (with a ground waste, grating, and sump) and connected to a public sewer or stormwater drain or to other approved disposal system complying with the Plumbing and Drainage Section of this Code - Part 4 Section 4, and it shall be paved with concrete or other non-absorbent material when required by the Building Authority.

3.418 Rear Yards

3.418.1 RESIDENTIAL AND INSTITUTIONAL BUILDINGS - At the rear of every building hereafter erected to be occupied as a one and two-family or multi-family dwellings (occupancy groups R-2 and R-3) or Institutional buildings (occupancy group I), there shall be maintained a yard of the minimum dimensions herein prescribed. When such yard serves as a required light and ventilation court, its minimum dimensions shall be those required for a court in this article.

3.418.2 DEPTH OF YARDS - The depth of a required yard between the extreme rear of the building and the rear lot line shall be not less than four and one-half metres (4.5 m) at any point for a height of ten metres (10.0 m), and shall increase one metre in depth for each additional three metres (3.0 m) of height above that limit; except that for a corner lot the minimum depth shall be not less than three metres (3.0 m). When the lot is less than twenty metres (20.0 m) in depth, the required yard may be diminished one metre (1.0 m) in depth for each two metres less than twenty metres (20.0 m).

3.418.3 OTHER OCCUPANCY GROUPS - In buildings of other occupancy groups, rear yards shall be provided to serve all habitable and occupiable rooms requiring light and ventilation from such source. The lowest level of such yards shall begin at the sill level of the first floor windows, with a depth of not less than three metres (3.0 m) for a height of ten metres (10.0 m) and shall increase one metre (1 m) for each additional four metres (4.0 m) of height above that level.

3.419 Obstruction of Courts and Yards

3.419.1 PERMISSIBLE PROJECTIONS - Every required court and yard shall remain unobstructed for its required area and full height, except for the projections permitted in Part 3 Section 2 of this Code. In residential and institutional buildings, clothes poles, arbors, garden trellises and other such accessories shall not be prohibited in the open spaces at ground level.

3.419.2 MOTOR VEHICLE PARKING - When approved by the Building Authority required court and yard areas may be used for automobile parking spaces or private garages not exceeding one storey in height when accessory to and only for the use of occupants of a residential building, provided required windows for light and ventilation are not obstructed thereby.

3.420 Fire Emergency Ventilating System(s)

3.420.1 COMMON CORRIDORS - In all buildings herein required to have fire emergency ventilating systems, the common corridors shall be constructed with:

(i) vertical fire-vent stacks and lateral fire-vent ducts as herein provided; or

(ii) windows to the outer air; or

(iii) mechanical ventilating or exhaust systems; or

(iv) other equivalent approved means for dissipating smoke, heated air and toxic gases directly to the outer air in the event of fire.

3.420.2 WHERE REQUIRED - Fire emergency ventilating systems shall be provided as decribed below:

1. In buildings used for I-1 and I-2 (institutional) occupancy groups which:

   (a) exceed three (3) stories or twelve metres (12.0 m) in height; and

(b) exceed nine hundred and fifty square metres (950 m^2) in floor area; and

(c) are occupied by more than fifty persons above the ground floor, or have more than twenty-five sleeping rooms above the ground floor.

2. In buildings used for R-1 and R-2 (hotel and apartment house) occupancy groups which:

(a) same as 1 (a) above;

(b) same as 1 (b) above;

(c) same as 1 (c) above.

3. In all fully enclosed industrial buildings without provision of exterior openings for ventilation purposes.

3.420.3 FIRE VENT DUCTS - When the common corridors and exitways are not ventilated by windows opening directly to the outer air as required in sub-section 3.414, a system of collecting fire ducts shall be provided in each storey of aggregate size to remove the smoke, hot air, and noxious fumes or gases in event of fire. Each duct shall be not less than one-tenth of one square metre (0.1 m^2) in area located in the common hallways with screened openings complying with the approved rules, constructed as provided for hot air ducts in Part 4, Section 1 of this Code. (Chimneys, Flues and Vent Pipes).

3.420.4 THERMOSTATIC OPERATION - When not connected to a vent stack, the inlet openings on each storey shall be controlled by automatic heat-operated devices as required in clause 3.416.3 above and in accordance with the approved rules.

3.420.5 FIRE VENT STACKS - When the fire ducts do not discharge directly to the outer air in each storey, one or more fire vent stacks of adequate capacity shall be installed to accommodate the discharge from the fire duct system in any one floor or enclosed fire area, but an individual stack shall not be less than four-tenths of one square metre (0.4 m^2) in area, and all stacks shall terminate in an approved automatic cowl or ventilator outlet above the roof.

3.420.6 LOCATION OF STACKS - The vent stack shall be located in as central a position as practicable with respect to the floor area vented thereby, preferably in the vicinity of vertical shafts and shall extend continuously to the roof.

3.420.7 VENT CONTROL OF STACKS - The vent control of the vertical stacks shall consist of approved non-combustible dampers, shutters, or a glazed metal sash designed to open outwardly, located not less than six metres (6.0 m) distant from window openings or exitway doors in adjoining walls and shall be equipped with a thermostatic unit arranged to open at a predetermined rate of temperature rise in accordance with the approved rules. Auxiliary mechanical means for manual operation of all vent controls shall be provided in an accessible location designated by the Building Authority.

3.420.8 STACK CONSTRUCTION - The stack enclosure shall be constructed to be vapour and smoke-tight with walls of not less than two (2) hour fire-resistance rating, and without openings other than the fire duct outlets and the top automatic ventilator outlet.

3.420.9 MECHANICAL EXHAUST SYSTEMS - When mechanical exhaust is required to operate the emergency ventilating system either in horizontal ducts or vertical vent stacks, the installation shall be thermostatically controlled and installed in accordance with the provisions of the Mechanical Code, and the approved rules.

3.421 Fire Ventilation of Open Wells

3.421.1 GENERAL - Open wells including unenclosed supplemental stairways and well openings for moving stairways not accepted as a required element of an exitway shall be permitted in buildings of other than occupancy groups A-4, and I (assembly, schools and institutional) when equipped with an approved automatic fire suppression system and protected on every floor pierced by the opening with an approved automatic exhaust system or by other approved method as herein required to prevent the passage of fire, smoke and gases to the storey above.

3.421.2 EXHAUST SYSTEM - The approved automatic exhaust system may be a separate unit or integrated with an approved air-conditioning system and shall be thermostatically controlled to operate simultaneously with the detection of fire.

3.421.3 CAPACITY OF EXHAUST SYSTEM - The exhaust system shall be of adequate capacity to create a down draft in the open well with sufficient velocity of flow over the entire area of the well opening under normal conditions of window and door openings in the building. In air-conditioned buildings, the system shall operate in a manner satisfactory to the Building Official with the normal air-conditioning fans shut off.

3.421.4 DRAFT STOP - An approved draft stop shall be installed at each storey of the open well. The draft stop shall enclose the perimeter of the unenclosed opening and shall extend from the ceiling downward at least one-half of one metre (0.5 m) on all sides. Automatic sprinklers shall be provided around the perimeter of the opening and within six-tenths of one metre (0.6 m) of the draft stop. The distance between the sprinkers shall not exceed two metres (2.0 m) centre to centre.

3.421.5 ELECTRICAL POWER - The electrical power for all parts of the exhaust system and fresh air intake shall be supplied from an emergency electrical system.

3.421.6 ALTERNATE PROTECTION - Unenclosed stairwells, when not protected as herein specified, shall be equipped with an approved automatic power-controlled fire shutter (metal roller door).

3.421.7 AIR-CONDITIONED BUILDINGS - The exhaust system herein required, when installed in an air-conditioned building, shall be so arranged as to automatically stop the operation of the mechanical air-conditioning and ventilating systems and close the dampers of the return air duct connection in the event of fire.

3.422 Window Cleaning Safeguards

3.422.1 GENERAL - All buildings over four stories in height or fifteen metres, whichever is the lesser, and in which the windows are cleaned from the outside shall be provided with anchors or safety rails or other approved safety devices approved by the Local Authority. Such anchors, safety belt terminals, or other devices shall be of an approved design, and constructed of non-corrosive materials securely anchored to the window frames or to walls of the building. Cast iron or bronze anchors shall be prohibited.

3.423 Sound Transmission Control (In Residential Buildings)

3.423.1 SCOPE - This section shall apply to all common interior walls, partitions and floor-ceiling construction between adjacent tenant units or between a tenant unit and adjacent public areas such as halls, corridors, stairs, or service areas in all residential occupancies.

3.423.2 AIRBORNE NOISE - Walls, partitions and floor-ceiling construction separating tenant units from each other or from public areas shall have a sound transmission class (STC) of not less than forty-five for airborne noise. This requirement shall not apply to dwelling unit entrance doors. However, such doors shall be tight fitting to the frame and sill.

3.423.3 TESTED ASSEMBLIES - All walls, partitions and floor-ceiling constructions tested in accordance with the applicable standard ASTM E90 (or equivalent British Standard) and which meet the requirements for a forty-five STC rating shall be considered as meeting the requirements of this Section.

3.423.4 STRUCTURE-BORNE SOUND - Floor-ceiling construction between tenant units and between a tenant unit and public service area within the structure shall have an impact insulation class (IIC) rating of not less than forty-five.

3.423.5 TESTED ASSEMBLIES - All floor-ceiling constructions tested in accordance with the applicable standard ASTM E492 (or equivalent British Standard) and which meet the requirements for a forty-five IIC rating shall be considered as meeting the requirements of this Section.

CARIBBEAN UNIFORM BUILDING CODE

PART 3
OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 5

MEANS OF EGRESS

PART 3

SECTION 5

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PART 3

SECTION 5

3.500 MEANS OF EGRESS

3.501 Scope

3.501.1 The purpose of this section of the Caribbean Uniform Building Code is to define minimum standards of exit/egress facilities for occupants of buildings.

3.502 General Requirements

3.502.1 All new buildings hereinafter erected and any existing buildings being altered or added to, or any existing buildings where the occupancy classification is being changed either with or without major alterations to the building, shall comply with this Code to provide a safe means of egress for the occupants in case of fire or other disaster.

3.502.2 When strict compliance with the provisions of this Code is not possible or practical, the Local Authority may accept alternate means of egress which will accomplish the same purpose.

3.502.3 No building under construction shall be occupied in whole or in part until all exit facilities required for the part to be occupied are completed and available for use.

3.503 Plans and Specifications

3.503.1 ARRANGEMENT OF EXITWAYS - The plans shall show in sufficient detail the location, construction, size and character of all exitways together with the arrangement of aisles, corridors, passageways and hallways leading thereto in compliance with the provisions of this Code.

3.503.2 NUMBER OF OCCUPANTS - In other than one and two-family and multi-family dwellings, the plans and the application for permit shall designate the number of occupants to be accommodated on every floor, and in all rooms and spaces as required by the building official. When not otherwise specified, the minimum number of occupants to be accommodated by the exitways shall be determined by the occupancy load prescribed in Part 3 Section 1 of this Code. The posted occupancy load of the building shall be limited to that number.

3.504 **Use and Occupancy Requirements**

3.504.1 NEW BUILDINGS - Every building and structure and part thereof hereafter erected shall have the prescribed number of exitways of one (1) or more of the approved types defined in this article. Exitways, in combination with the exitway access and exitway discharge, shall provide safe and continuous means of egress to a street or to an open space with direct access to a street.

3.504.2 MIXED USE GROUPS - In buildings classified in more than one (1) use group, each fire area shall be considered separately in determining the required number, capacity, size and construction of all exitways.

3.504.3 MULTIPLE TENANTS - When more than one (1) tenant occupies any one (1) floor of a building or structure, each tenant shall be provided with direct access to approved exitways.

3.504.4 The number of occupants of any building, or portion of any building shall not exceed the permitted or posted capacity based on occupancy loading.

3.504.5 When assessing occupancy loading of any building Part 3 Section 1 of this Code shall be read in conjunction with this section (5).

3.505 **Air-Conditioned Buildings**

3.505.1 LOCATION OF STAIRWAYS - In all buildings, without exterior window openings in all stories, that are artificially ventilated and air-conditioned as provided in Part 3 Section 4, the stairways element of required exitways shall be located so as to be accessible to the Fire Department either through the access openings specified in Part 3 Section 2 or as otherwise approved in at least alternate stories of the building.

3.505.2 EXHAUST DUCTS - Exhaust ducts or vents of air-conditioning systems shall not discharge into stairway or elevator enclosures, nor shall corridors serving as exitway egress be used as the return exhaust from air-conditoned spaces through louvres or other devices in the doors or partitions enclosing such air-conditioned spaces; unless such passageways are equipped with approved smoke detectors to automatically stop the supply and exhaust fans and close the louvres, and unless such use is approved by the Building Authority.

3.506 **Existing Buildings**

3.506.1 OWNER RESPONSIBILITY - The owner or lessee of every existing building shall be responsible for the safety of all persons in, or occupying, such premises with respect to the adequacy of means of egress therefrom.

3.506.2 UNSAFE OR INADEQUATE MEANS OF EGRESS - In any existing building, not provided with exitway facilities as herein prescribed for new buildings and in which the exitways are deemed inadequate for safety by the building official or safety inspector, such additional provision shall be made for safe means of egress as he shall direct.

3.506.3 APPEAL AGAINST ORDER TO PROVIDE ADDITIONAL EXITS -Within fourteen (14) days of an order to update the means of egress from any building, an owner or lessee may lodge an appeal with the Local Authority against complying fully with that order. The Local Authority shall then appoint an Arbitrator, who along with representatives of the Fire Board and others will again look into ways and means of bringing such building up to a reasonable safety standard. The Arbitrator's decision will be final and binding on all parties.

3.507 **Maintenance of Exitways**

3.507.1 It shall be unlawful to obstruct; or reduce in any manner, the clear widths of any doorway, hallway, passageway, stairway or any other exitway required by the provisions of this Code.

3.507.2 All exterior stairways and fire escapes shall be properly painted before and after erection; and shall be scraped and painted as often as necessary to maintain them in safe condition. Particular care must be taken to ensure treads of stairs and decking of walkways shall not be allowed to become smooth and slippery during wet weather.

3.508 **Occupancy Load**

3.508.1 DESIGN OCCUPANCY LOAD - In determining required exitway facilities the number of occupants for whom exitways facilities shall be provided will be in accordance with the following:

a. the actual number of occupants for whom each occupied space, floor, or building, as the case may be, is designed; or

b. the number of occupants computed at the rate of one (1) occupant per unit of area as prescribed in Table 3.508 or

c. the number of occupants of any space as computed in a or b above, plus the number of occupants similarly computed for all spaces that discharge through the space in order to gain access to an exitway.

3.508.2 ASSEMBLY OCCUPANCY - The occupancy load for places of assembly may be determined as provided in 3.508.1 above if the necessary aisles and means of egress are provided as approved by the building authority. An aisle, egress and seating diagram may be required by the Building Authority to substantiate the occupancy load.

3.508.3 MEZZANINE LEVELS - The occupancy load of a mezzanine floor discharging through a floor below shall be added to that floor occupancy, and the capacity of the exitways shall be designed for the total occupancy load thus established.

3.508.4 ROOFS - Roof areas occupied as roof gardens or for assembly, storage or other purposes shall be provided with exitway facilities to accommodate the required occupancy load, but there shall not be less than two (2) approved means of egress for assembly uses from such roof areas.

3.508.5 SPECIAL OR UNLISTED OCCUPANCIES - Where data regarding the area per person for an occupancy is not listed in Table 3.508, the occupant load shall be established by the architect or engineer, subject to the approval of the Building Authority.

3.508.6 CONFLICTS - When there are special requirements for specific occupancies or uses which differ from general requirements herein prescribed, such special provisions shall take precedence.

3.508.7 NON-SIMULTANEOUS OCCUPANCY - The occupancy load of toilets, locker rooms, meeting rooms, storage rooms, employee cafeterias, and similar rooms or spaces that are not occupied at the same time as other rooms or spaces on the same floor of a building, may be omitted from the occupant load calculation of the floor on which they are located, to the extent that such spaces only serve occupied rooms on the same floor.

3.508.8 MODIFICATIONS - The following modificaton, alterations, dispensations, may be made to the occupancy load of any building by the Building Authority.

(a) When the actual occupancy load of any space will be significantly different from that determined by Table 3.508 the Building Authority may establish an alternate basis for the determination of the occupant load. The space occupied by permanent fixtures or displays could serve to reduce the occupant load - for example

(b) When a building is altered or changed in occupancy so as to require enlarged exitway facilities, the building authority may authorize the alteration or change in occupancy, without an enlargement of exitway facilities, provided the occupancy load is limited to that accommodated by the existing exitway facilities as determined by the provisions of this Code.

3.509 Types and Locations of Exitways

3.509.1 GENERAL - All approved exitways, including doorways, passageways, corridors, interior stairways, exterior stairways, moving stairways, smokeproof enclosures, ramps, horizontal exits, bridges, balconies, fire escapes and combinations thereof shall be arranged and constructed as provided in this Code.

3.509.2 ARRANGEMENT - All required exitways shall be located as to be discernable and accessible with unobstructed access thereto and so arranged as to lead directly to the street or to an area of refuge with supplemental means of egress that will not be obstructed or impaired by fire, smoke or other cause.

3.509.3 EXITWAY DISCHARGE - All exitways shall discharge directly on to a public way or at a yard, court or open space of the required width and size to provide all occupants with a safe access to a public way.

3.509.4 REMOTE LOCATION - Whenever more than one (1) exitway is required from any room, space or floor of a building, they shall be placed as remote from each other as practicable, and shall be arranged to provide direct access in separate directions from any point in the area served.

3.509.5 LENGTH OF TRAVEL - Except as modified by provisions of 3.511.3 of this Code for buildings with one (1) exitway, all exitways shall be so located that the maximum length of exitway access travel, measured from the most remote point to an approved exitway along the natural and unobstructed line of travel shall not exceed the distance given in Table 3.509; except where the area is subdivided into rooms or compartments, and the egress travel in the room or compartment is not greater than 16 metres (16 m) in use groups equipped with an automatic fire suppression system, the distance shall be measured from the exit way access entrance to the nearest exitway.

TABLE 3.508

MAXIMUM FLOOR AREA ALLOWANCES PER OCCUPANT
IN SQUARE METRES

OCCUPANCY GROUP	USE	AREA PER OCCUPANT	
ASSEMBLY: (A)	concentrated-chairs only, not fixed	$0.65m^2$	net
	unconcentrated-tables and chairs	1.4	net
	standing spaces(s)	0.28	net
	area with fixed seats	no. of fixed seats	
	school/college classrooms	2.0	net
	library reading rooms	4.6	net
	library stack areas	9.0	gross
BUSINESS: (B)	General business areas	9.0	gross
	court rooms	3.75	net
FACTORY AND INDUSTRIAL: (F)	industrial areas generally	18.0	gross
INSTITUTIONAL: (I)	sleeping areas	7.5	gross
	in-patient treatment areas	22.5	gross
	out-patient areas	9.0	gross
MERCANTILE: (M)	shops & other vocational room areas	4.6	net
	salesrooms - shopping malls	4.6	net
	mercantile basement & ground floors	2.8	gross
	mercantile upper floors	5.6	gross
	storage, packing & shipping areas	9.0	gross
RESIDENTIAL: (R)	houses, flats, condominiums (R-3+4)	18.0	gross
	motels, boarding houses, hostels, dormitories (R-2)	9.0	gross
STORAGE: (S)	storage areas generally, also mechanical equipment rooms	27.0	gross

TABLE 3.509

LENGTH OF EXITWAY ACCESS TRAVEL IN METRES

OCCUPANCY GROUP	WITHOUT FIRE SUPPRESSION SYSTEM	WITH A FIRE SUPPRESSION SYSTEM
ASSEMBLY (A)	45	60
BUSINESS (B)	60	90
FACTORY AND INDUSTRIAL (F)	60	90
HIGH HAZARD (H)	-	22
INSTITUTIONAL (I)	30	60
MERCANTILE (M)	30	45
RESIDENTIAL (R)	30	45
STORAGE, LOW HAZARD (S.1)	90	120
STORAGE, HIGH HAZARD (S.2)	60	90

NOTE: The maximum length of exitway access travel in unlimited area buildings shall be 120 metres.

3.510 Capacity of Exits

3.510.1 UNIT OF EGRESS WIDTH - The unit of egress width for all approved types of means of egress parts and facilities shall be 560 mm (22 inches) with a credit of one-half (1/2) unit for each 300 mm (12 inches) width in addition to one (1) or more 560 mm units. Fractions of a unit of width less than 300 mm shall not be credited.

3.510.2 DESIGN ALLOWANCE FOR USE GROUPS - Except as may be specifically modified in Part 3 Section 3, the design capacity per unit of egress width shall be computed in accordance with Table 3.510 for the specified use or occupation group.

TABLE 3.510

CAPACITY PER UNIT EGRESS WIDTH

OCCUPANCY GROUP	Without fire suppression system Number of Occupants		With fire suppression system Number of Occupants	
	Stairways	Doors, Ramps and Corridors	Stairways	Doors, Ramps and Corridors
Assembly (A)	75	100	113	150
Business (B)	60	100	90	150
Factory and Industrial (F)	60	100	90	150
High Hazard (H)	-	-	60	100
Institutional (I)	22	30	33	45
Mercantile (M)	60	100	90	150
Residential (R)	75	100	113	150
Storage (S)	60	100	90	150

3.511 **Number of Exitways**

3.511.1 GENERAL - The following general requirements apply to buildings of all use groups. More restrictive requirements that may be provided in Part 3 Section 3 for special uses and occupancies shall take precedence over the general provisions of this section.

3.511.2 MINIMUM NUMBER - There shall be not less than two (2) approved independent exitways serving every storey except in one and two-family dwellings and as modified in 3.511.3 below.

3.511.3 BUILDINGS WITH ONE EXITWAY - Only one (1) exitway shall be required in buildings of the use group and characteristics specified in the following Table 3.511 and in the first storey of buildings 190 square metres or less in area with an occupancy load not exceeding fifty (50) persons on the first storey.

TABLE 3.511

BUILDINGS WITH ONE EXIT

Characteristics Of The Building

Occupancy Group	Max. height above ground Level	Size	Max. exit-way access travel distance	Min. Fire-resistance rating of exitway enclosure	Min. Fire-resistance rating of opening protection
R-2 (residential multi-family)	2 stories	4 dwelling units per floor	15 metres	1 hour	1 hour
B (Business)	2 stories	280 m^2 per floor	22 metres	1 hour	1 hour

Note 1: Areas complying with definition for basements shall not be counted as a storey.

3.511.4 EMERGENCY ESCAPES - Every sleeping room below the fourth storey shall have at least one openable window or exterior door approved for emergency egress or rescue. These emergency exits must be operable from the inside without the use of separate tools. Where windows are approved as a means of egress or rescue, they shall have a sill height of not more than 1.2 metres above the floor.

3.511.5 All egress or rescue windows from sleeping rooms must have a minimum net clear opening of six-tenths of one square metre (0.6 m^2). The minimum net clear opening height shall be 610 millimetres and the minimum net clear opening width dimension shall be 510 millimetres.

3.511.6 EXCEPTION - Ground floor windows may have a minimum net clear opening of one-half of one square metre (.5 m^2).

3.511.7 OPEN PARKING STRUCTURES - Parking structures shall have not less than two (2) exitways from each parking tier, except that where vehicles are mechanically parked, only one (1) exitway need be provided. The maximum distance from any point on a parking tier to an exitway at that tier shall not exceed thirty metres (30 m). Unenclosed vehicle ramps may be considered as required exitways if pedestrian facilities are provided. Interior exitway stairways need not be enclosed.

3.512 Passages and Corridors as Exitway Accesses

3.512.1 ACCESS PASSAGEWAYS - Direct exitways access shall be provided to required exitways through continuous passageways, aisles or corridors, conveniently accessible to all occupants and maintained free of obstruction.

3.512.2 TURNSTILES AND GATES - Access through turnstiles, gates, rails or similar devices shall not be permitted.

3.512.3 RESTRICTIONS - The required width of passageways, aisles or corridors shall be maintained free of projections and restrictions except doors opening into such spaces may reduce the clear width to not less than one-half (1/2) the required width. When fully open the door may project not more than 150 mm into the required width.

3.512.4 DEAD ENDS - Exitways access passageways and corridors in all stories which serve more than one (1) exitway shall provide direct connection to such exitways in opposite directions from any point in the passageway or corridor, insofar as practicable. The length of a dead end corridor shall not be more than six metres (6.0 m).

3.512.5 WIDTH - The unit of egress width and occupancy allowances of aisles and corridors, unless otherwise provided for special uses and occupancies in Part 3 Section 3, shall comply with Table 3.510 with a minimum total width of 1.1 metres except in institutional (I) buildings used for the movement of beds which shall be 2.4 m; in schools with more than one hundred (100) occupants which shall be 1.85 m; in one and two-family dwellings which shall be one metre (1.0 m); and in churches and chapels, side aisles shall be not less than eight-tenths of one metre (0.8 m) clear.

3.512.6 ENCLOSURES - All corridors serving as exitways access shall be enclosed in fire separation walls having a fire-resistance rating of at least one (1) hour when serving an occupancy load greater than thirty (30).

3.512.7 OPENING PROTECTIVES - All door assemblies from rooms opening onto a corridor required to be of one (1) hour fire-resistance rated construction shall be self-closing (or automatic closing by smoke detection) with a thirty (30) minute (1/2 hour) fire protection rating without the hose stream, and labelled and listed by an independent, approved agency.

3.512.8 All doors or door assemblies from rooms opening on to a corridor, required by Table 3.114 (Part 3 Section 1) to be of two hour fire-resistance rated construction, shall be one and one-half (1 1/2 hr) hour fire doors.

3.513 Ground Floor Passageway and Lobbies Used as Exitways

3.513.1 PASSAGEWAYS - Every required interior and/or exterior exitway element which does not adjoin a public way (street, footpath, right-of-way or lane) shall be directly connected to a public way or to an open court leading to a public way by an enclosed and protected passageway.

3.513.2 VESTIBULE - An exitway may discharge into an interior vestibule used for ingress and egress only and which complies with the following:

(a) the vestibule depth from the exterior of the building is not greater than 3 metres and the width is not greater than six metres (6.0 m) and

(b) the vestibule is separated from the remainder of the level of discharge by self-closing doors and the equivalent of six and one-half millimetres (6.5 mm) thick wired glass in steel or aluminium frames.

3.513.3 LOBBY - An exitway may discharge into an interior lobby which shall be provided with an automatic fire suppression system and any other portion of the floor with access to the lobby shall be provided with an automatic fire suppression system or shall be separated therefrom in accordance with the requirements for the enclosure of exitways.

3.513.4 WIDTHS AND HEIGHTS - The effective width of the passageway shall be not less than three-quarters (3/4) of the aggregate width of all required exitway stairways leading thereto and all required exitway doorways opening into the passageway. Such passageway shall have a minimum width of 1.1 metres and a minimum clear ceiling height of 2.4 metres.

3.513.5 MAXIMUM STAIRWAY LIMITATIONS - Not more than fifty (50) percent of the required stairways shall discharge through the same passageway.

3.514 Means of Egress Doorways

3.514.1 GENERAL - The requirements of this section shall apply to all doorways serving as a component or element of a means of egress; except that this section shall not apply to doorways leading to or from required stairways (see clauses 3.517.17, 3.519.4 and 3.520.40).

3.514.2 NUMBER OF DOORWAYS - Every room or tenant space with an occupancy load of more than fifty (50) or which exceeds two hundred square metres (200 m^2) in area shall have at least two (2) egress doorways leading from the room or tenant space to an exitway or corridor. All doors shall swing in the direction of egress travel when serving an occupancy load of twenty-five (25) or more or a high hazard occupancy.
Exception: One and two-family dwellings.

3.514.3 ENTRANCE AND EGRESS DOORWAYS - Where separate doors are provided for entrance and egress use, the entrance door shall be clearly marked "Entrance only" in letters not less than 150 mm in height and be legible from both inside and outside.

3.514.4 SIZE OF DOORS - The minimum width of single door openings shall provide a clear width of not less than eight hundred and ten millimetres except in one and two-family dwellings (use groups R-3 and R-4) the clear width shall be not less than seven hundred and ten millimetres (710 mm). The maximum width shall be 1.2 m nominal. Means of egress doors in institutional buildings (use group 1) used for the movement of beds shall be at least 1.1 m wide. When the doorway is subdivided into two (2) or more separate openings, the minimum clear width of one (1) opening shall be not less than 810 mm, and each opening shall be computed separately in determining the number of required units of egress width. The height of doors shall not be less than two metres (2.0).

3.514.5 LOCATION OF DOORS - The required doors opening from a room or space within a building and leading to an exitway access shall be located as far as practicable from each other. The distance of exitway access travel from any point in a room or space to a required exitway door shall not exceed the limitations of clause 3.509.5 of this Code (see Table 3.509).

3.514.6 DOOR HARDWARE - All egress doors shall be readily openable from the side from which egress is to be made without the use of a key or special knowledge or effort except for special institutional uses as indicated in 3.514.9. Except for dwelling units, draw bolts, hooks and other similar devices shall be prohibited on all egress doors, unless there is a readily visible, durable sign on the door stating "This door to remain unlocked during occupancy". The sign shall be in letters not less than 25 mm high on a contrasting background. The locking device must be of a type that will be readily distinguishable as locked. The use of manually operated flush bolts or surface bolts is prohibited.

3.514.7 Double cylinder dead locks (or bolts) requiring a key operation on both sides are prohibited on required means of egress in all occupancies.

3.514.8 PANIC BOLTS/DEVICES - All doors equipped with latching devices, in buildings of occupancy group A (assembly) with an occupant load greater than forty-nine (49) shall be equipped with approved panic hardware. Acceptable panic hardware will be a device which causes the door latch to release when a force of 7 kilograms is applied in the direction of egress to a bar or panel extending not less than one-half (1/2) of the width of the door and at a height greater than 0.75 metres but less than 1.1 metres above the floor.

3.514.9 REMOTE CONTROL - In rooms of use group I-1 (institutional, restrained) occupied as places of detention, approved releasing devices with remote control shall be provided for emergency use.

3.514.10 MECHANICAL OPERATIONS - All doors which open into enclosed exitway stairs, exitway passageways or those which are installed to provide fire or smoke barriers across corridors shall be self-closing and be so maintained, or shall be automatic doors which will close upon activation of an approved smoke detector. Where egress doors are arranged to be opened by non-power-operated mechanical devices of any kind, they shall be so constructed that the door may be opened manually and will release under a total pressure of not more than 7 kilograms applied in the direction of egress travel. Power-operated exitway doors shall be capable of being opened with not more than 22.5 kilograms pressure applied at the normal door knob location when power is lost.

3.514.11 DOOR CONSTRUCTION - All required egress doors that serve as an element of an exitway shall be self-closing or automatic except for ground-floor exitway discharge doors and revolving exitway doors.

3.514.12 GROUND-LEVEL EXITWAY DISCHARGE DOORS - Doors on ground floors may be glazed with plate glass not less than 6.5 millimetres plate glass thick, or with any other approved glazing materials. Approved doors having one (1) or more unframed edges may be used, provided they are constructed of safety glazing not less than 12.5 mm thick.

3.514.13 DOORWAY EXIT RAMPS: FOR THE PHYSICALLY HANDICAPPED - From each ground-floor exitway required by Part 3 Section 2 for the physically handicapped and aged, there shall be provided, after exiting the structure, a hard surfaced area a minimum of 1.2 metres in width centered at the required opening and extending to a property line adjoining a public street or alley or a point which is a minimum of 3 metres clear from any part of the structure. At the exit door, the level of the exterior surface shall not be elevated from the floor inside the door, nor shall it be more than 50 millimetres below the floor inside the door. A ground-floor exitway shall not have a threshold greater than 12 millimetres high, with bevelled edges. The floor and exterior surface shall not have a grade of more than two (2) percent for a distance of 1.8 metres either side of the door. The remainder of the required exterior hard surfaced area shall have an elevating gradient not greater than five (5) percent. Such walks shall be of a continuing common surface not interrupted by steps or abrupt changes in grade.
<u>Exception</u>: One and two-family dwellings (occ. groups R-3 and R-4) and occ. group T.

3.514.14 EXIT DOOR ARRANGEMENT - Hinged outward opening doors in series shall not be regarded as approved exit doors unless they have at least 2 metres space between them when measured in their closed position.
<u>Exception</u>: (a) Power-operated doors
(b) One and two-family dwellings (occupancy groups R-3 and R-4)
(c) Buildings of occupancy group T (Temporary Structures).

3.514.15 REVOLVING DOORS - Generally, revolving doors, shall not be used as emergency exit doors, but a special dispensation may be applied for if the following conditions are met (see 3.514.16, a and b).

3.514.16 (a) When hinged and outward opening exit doors are placed on either side of a revolving door and these additional doors make up at least two-thirds of the required exitway units, and

(b) When the revolving doors can have two alternate leaves hinged onto the two remaining leaves and such folded together leaves automatically lock in a position ninety degrees (90°) from the main line of wall in which doors are situated, then such revolving doors may be counted as one exit unit.

3.515 Horizontal Exits

3.515.1 GENERAL - Horizontal exits as herein defined shall be accepted as an approved element of a required means of egress when complying with the requirements of this section of the Code. The connection between the areas of refuge as herein specified may be accomplished by protected openings in a fire-resistance rated wall, by a vestibule, or by an open-air balcony or bridge.

3.515.2 SEPARATION - The separation between fire areas shall be provided by at least a two (2) hour fire-resistance rated fire wall or fire separation wall.

3.515.3 OPENING PROTECTIVES - All fire doors in horizontal exits are to be self-closing or automatically closing when activated by an approved smoke detector. All doors shall swing in the direction of egress travel. When serving as a dual element of a means of egress, there shall be adjacent openings with swinging fire doors opening in opposite directions.

3.515.4 SIZE OF DOORS - Size of openings in fire walls shall comply with the provisions of Section 6 but the width of one (1) opening used as a required exit shall not be greater than two and one-quarter metres (2.25 m) nor shall the area exceed seven and one-half square metres (7.5 m^2).

3.515.5 AREAS OF REFUGE - The discharge area of a horizontal exit shall be either public areas or spaces occupied by the same tenant and each such area of refuge shall be adequate to house the total occupancy load of both connected areas. The capacity of areas of refuge shall be computed on a net floor area allowance of three-tenths of one square metre (0.3 m^2) for each occupant to be accommodated therein except for non-ambulatory institutional areas which shall be ten times this amount per occupant, not including areas of stairs, elevators and other shafts or courts.

3.515.6 UNLOCKED DOORS - Horizontal exit doors shall be kept unlocked and unobstructed whenever the area on either side of the horizontal exit is occupied.

3.515.7 EGRESS FROM AREA OF REFUGE - This shall be by stairway (s). In multi-storey buildings, there shall be at least one (1) interior enclosed stairway or smoke-proof enclosure on each side of the horizontal exit, and any fire area not having a stairway accessible thereto shall be considered as part of an adjoining section with such stairway; but the length of exitway access travel distance to the horizontal exit or the required exitway shall not exceed the requirements laid down in 3.509.5 and Table 3.509 of this Code.

3.516 Egress Ramps

3.516.1 CAPACITY - The capacity of ramps used as an egress component shall be computed in accordance with sub-section 3.510 to and including Table 3.510 of this Code.

3.516.2 MINIMUM DIMENSIONS - The minimum dimensions of all egress ramps shall be as required in 3.517.3 and 3.517.5 inclusive.

3.516.3 WIDTH - The minimum width of an egress ramp shall be not less than that required for passageways/corridors in clause 3.513.4.

3.516.4 HEADROOM - The minimum headroom in all parts of the egress ramp shall be not less than two metres (2.0 m).

3.516.5 RESTRICTIONS - Egress ramps shall not reduce in width in the direction of egress travel. Projections into the required ramp and landing width are prohibited except for handrails and stringers. Doors opening onto a landing shall not reduce the clear width to less than one and one-tenth metres (1.1 m).

3.516.6 LANDINGS - Landings shall be provided at all points of turning, entrance, exiting and doors. Ramp slopes greater than one (1) in fifteen (15) shall have landings at the top, bottom and each 1.2 metres (5 feet) of vertical rise. Each landing shall have a minimum length of 1.8 metres except the bottom landing which shall have a length of two metres (2.0m).

3.516.7 MAXIMUM SLOPE - A ramp used for egress for the physically handicapped shall have a maximum slope of one (1) in twelve (12). All other egress ramps shall have a maximum slope of one (1) in eight (8).

3.516.8 SURFACE - For all slopes exceeding one (1) in twelve (12), and wherever the use is such as to involve danger of slipping, the ramp shall be surfaced with approved non-slip materials.

3.516.9 HANDRAILS - Handrails shall be provided on at least one (1) side of every ramp having a slope greater than one (1) in fifteen (15), and they shall be not less than 760 millimetres nor more than 900 millimetres in height, measured from the surface of the ramp. Handrails shall be smooth and shall extend 300 millimetres beyond the top and bottom of the ramp and return to walls or posts at the ends.

3.516.10 RAMP CONSTRUCTION - Ramps used as an exitway shall conform to the applicable requirements of clauses 3.517.23 and 3.517.24 as to materials of construction and enclosure.

3.517 Interior Exitway Stairways

3.517.1 CAPACITY - The capacity of stairways and doors per unit of exit width shall be computed in accordance with sub-section 3.510.

3.517.2 MINIMUM DIMENSIONS - These shall be as required below under 3.517.3 to 3.517.5 inclusive.

3.517.3 WIDTH - All interior exitway stairways shall be not less than 1.1 metres in width, except that such width may be reduced to one metre (1.0 m) when serving an occupancy load of fifty (50) or less.

3.517.4 HEADROOM - The minimum headroom in all parts of the stair enclosure shall be not less than two metres (2.0 m) vertically from the tread nosing or from the floor surface of the landing or platform.

3.517.5 RESTRICTIONS - Stairways shall not reduce in width in the direction of exit travel. Projections into a stairway are prohibited except for handrails and for stairway stringers which may project not more than forty millimetres.

3.517.6 LANDINGS AND PLATFORMS - The least dimension of landings and/or platforms shall not be less than the required width of the stairway.

3.517.7 VERTICAL RISE - In no building shall a stairway have a height of vertical rise greater than three and one-half metres (3.5 m).

3.517.8 TREADS AND RISERS - The minimum widths of treads and the maximum heights of risers shall be as indicated in the following Table 3.517.

TABLE 3.517
TREAD AND RISER SIZES/DIMENSIONS

Occupancy Group	Maximum for Risers		Minimum for Treads*	
Assembly (A)	190 mm	7 1/2 inches	250 mm	10 inches
Institutional (I)	190 mm	7 1/2 inches	250 mm	10 inches
One & Two-family Dwellings (R-3 & R-4)	210 mm	8 1/4 inches	230 mm	9 inches
All others	203 mm	8 inches	230 mm	9 inches

* Excluding nosings

3.517.9 WINDERS - Winders shall not be permitted in required exitway stairways except in one and two-family dwellings and stairways serving a single dwelling unit. Such winders shall have a tread width of not less than 230 mm at a point one-third of the width of the stair in from the narrower end of the winder. Also the narrowest of tread allowable is 127 mm.

3.517.10 STAIRWAY GUARDS AND HANDRAILS - Stairways shall have continuous guards and handrails on both sides, and in addition thereto, stairways more than two and three-tenths metres (2.3 m) in required width shall have intermediate handrails dividing the stairway into two or more lanes (each with handrails). Stairways in one and two-family dwellings may have one (1) handrail.

3.517.11 HANDRAIL REQUIREMENTS - These are as follows:

(a) Handrails may project not more than ninety millimetres (90 mm) into the required stair width.

(b) Handrails shall be not less than 760 millimetres nor more than nine hundred millimetres (900 mm) measured vertically, above the nosing of the treads.

(c) Handrails shall extend one-half of one metre (0.5 m) beyond the top and bottom step if a guard or wall exists and shall be returned to walls or posts at the ends of the stairways.

(d) Handrails shall be designed to withstand an applied load of one hundred (100 kilos) in any direction at any point.

3.517.12 STAIR GUARDS - These are often referred to as dwarf walls, or protective walls to which a handrail can be attached where a stairway or flight of stairs is not adjacent or attached to a wall of the building. A stairguard is not required on both sides of a free standing stair, provided that the handrail has an infill of balusters, steel mesh, protective wood, or metal rails parallel to and below the handrail.

3.517.13 Guards shall not be less than 1.1 metres in height measured vertically above the nosing of the stair tread(s). (See 3.517.14 for exceptions to this).

3.517.14 Stair guards may be 760 millimetres or more in height provided any one of the following requirements apply:

(a) The stairway does not exceed six metres in height.

(b) The stairway reverses direction at each landing and that the distance between the strings of up flight and down flight of stairs does not exceed 130 mm.

(c) The stairway (s) are in R-3 or R-4 occupancies where guards of 900 mm are permitted.

3.517.15 CONSTRUCTION OF STAIR GUARDS - Guards need not necessarily be of solid walling, but they must not have any openings higher or wider than 150 millimetres; they may therefore be constructed of firm framing with infill of pierced (ornamental) concrete or clay blocks, panels of heavy wire mesh/or expanded metal, or ornamental metal grilles, or other protective equivalent or other protection required under 3.517.12 for handrails.

3.517.16 Guards are required full height along open sided floor areas, mezzanine floors, and/or landings except in the case of R-3 or R-4 occupancies where the reduced height of 900 mm is permitted.

3.517.17 STAIR EXITWAY DOORS

3.517.18 WIDTH - The width of every exitway door to or from a stairway shall be not less than the number of units of exit width required for the capacity of the stairway which services the floor or area from which the exitway door leads; but such a door shall not be less than seven hundred and ten millimetres (710 mm) in clear width in use group R-3 buildings (one and two-family dwellings) nor less than 810 millimetres in clear width in all other use groups.

3.517.19 DIRECTION OF SWING - All doors shall swing on a landing in the direction of exit travel. When opening, stair exitway doors shall not reduce the width of landings to less than one-half (1/2) the minimum required for its capacity. When fully open, the exitway door shall not project more than 150 mm onto the landing.

3.517.20 DOOR CONSTRUCTION/FIRE RATING - All doors including frames, architraves or other door protectives shall have a one hour fire rating, shall be approved self-closing, swing fire doors, except for occupancies R-3 and R-4 where normal thickness solid CORE doors with a half-hour rating are permitted, provided they open outwards.

3.517.21 CIRCULAR AND/OR SPIRAL STAIRWAYS - Both circular and spiral stairways are permitted as approved means of egress provided that:

(a) they are constructed of non-combustible materials,

(b) are only used in R-3 and R-4 occupancies or in other occupancies that are not more than 2 stories in height,

(c) or are only serving a mezzanine floor of not greater area than twenty-five sq. metres with an occupancy loading of ten or less,

(d) the minimum width of such stairway is not less than 700 millimetres.

3.517.22 TREADS - The tread widths of spiral stairways shall be not less than two hundred millimetres at a point three hundred millimetres in from narrow end; and in the case of circular stairways the minimum width of tread at the narrow end shall be 230 millimetres.

3.517.23 STAIRWAY CONSTRUCTION - Except in the cases of:

(a) R-3 and R-4 occupancy (Residential) or

(b) not more than two storey construction in low hazard occupancies (such as S-2), or where both floors are relatively open such as larger shops or mezzanine floors in shopping or business areas; where stairways of timber are permitted,

all stairways shall be constructed of non-combustible materials with solid risers, treads, landings, and platforms and with non-slip, non-combustible floor coverings. These stairs, however, may have wooden handrails.

3.517.24 STRENGTH - All stairways, platforms, landing and exitways in other than one and two-family dwellings (R-3 and R-4 occupancies) shall be adequate to support a live load of 488 kg/m^2 and a concentrated point load of 150 kilograms.

3.517.25 ENCLOSURES - Required interior exitway stairways shall be enclosed in fire separation construction with the fire-resistance ratings as required in Occupancy and Construction Classification, Part 3 Section 1 of this Code and as given in Table 3.114. No doors shall open into such stairway enclosure except required exitway doors.

3.517.26 COMBUSTIBLE CONSTRUCTION - In all buildings of types 3 and 4 construction (see under occupancy and construction classification Part 3 Section 1 of this Code, and Table 3.114), the stairways and their enclosures may be constructed of timber or other approved materials of similar characteristics and of adequate strength.

3.517.27 COMMUNICATION FLOORS - In any building, other than occupational groups A-4 (assembly; schools) or I (institutional), with low hazard occupancy (use group S-2), or with ordinary hazard occupancy (groups B, M, R-1 and R-2) with automatic sprinkler protection and where necessary to the effective utilization of a building site with sloping groundline or otherwise essential to the functional design of the building, not more than three (3) communicating floor levels may be permitted without enclosure or protection between such areas, only provided all the conditions described below are met.

(a) the lowest, or next to the lowest, level is a street level floor;

(b) the entire area, including all communicating floor levels, is sufficiently open and unobstructed to be assumed that a fire or other dangerous condition in any part will be immediately obvious to the occupants of all communicating levels and areas;

(c) egress capacity is simultaneously sufficient for all the occupants of all communicating levels and areas, all communicating levels in the same fire area being considered as a single floor area for purposes of determination of required egress capacity; and

(d) each floor level, considered separately, has at least one-half (1/2) of its individual required egress capacity provided by an exitway or exitways leading directly out of that area without traversing another communicating floor level or being exposed to the spread of fire or smoke thereafter.

3.517.28 DISCHARGE IDENTIFICATION - Stairways which continue beyond the floor of discharge shall be interrupted at the floor of discharge by partitions, doors or other effective means of preventing persons from continuing past the floor of discharge while egressing. A sign shall be provided at each landing in all interior stairways more than three (3) stories in height designating the floor level above the floor of discharge.

3.518 **Access to Roofs**

3.518.1 BY STAIRWAY OR LADDER - In buildings more than three (3) stories in height except those with a roof slope greater than one (1) in three (3), access to the roof shall be provided by means of a stairway or a ladder and trap door; the ladder shall not be on the exterior of the building. Where the roof is used as a roof garden or for other habitable purposes, sufficient stairways shall extend to it to provide the necessary exitway facilities from the roof as required for such occupancy.

3.518.2 OPTIONAL STAIRWAY OR LADDER - Buildings not required to have a stairway or ladder to the roof as described above, may include such a stairway or ladder at the discretion of the designer of the building. The stairway or ladder shall conform to the provisions of this section, except that ladders may be placed on the exterior of the building. The side members of exterior ladders shall be carried over the coping or parapet to afford hand hold; the ladder shall be metal, other design details of such exterior ladders are subject to the approval of the building official.

3.518.3 ROOF ENCLOSURES - Stairways extending through roofs shall be enclosed in roof structures of fire-resistance rated construction meeting the requirements of this Code.

3.519 **Smoke-Proof Enclosures - (Smoke-Proof Tower Escapes)**

3.519.1 GENERAL - A smoke-proof enclosure shall consist of a continuous stairway, enclosed from the highest point to the lowest point, meeting the requirements of this section.

3.519.2 WHERE REQUIRED - At least one (1) of the required exitways shall be a smoke-proof enclosure in buildings over six (6) stories or twenty-four metres in height when of one (1) of the following occupancy groups:

(a) groups A-2, A-3, A-4, A-5 (assembly other than theatres)

(b) group B (Business)

(c) group F (Factory and industrial)

(d) group I (Institutional)

(e) group M (mercantile); and

(f) group R1 (residential, hotel).

3.519.3 ACCESS - Exitway access to the stairway at each story shall be through a vestibule or balcony with an unobstructed width not less than the required stairway width and a minimum dimension of two metres (2.0 m) in the direction of travel.

3.519.4 DOORS INTO - Door openings from interior spaces to the vestibule or balcony and from the vestibule or balcony to the stairway, shall be as required in clause 3.514.4. The doors from interior spaces to the vestibule shall have a fire-resistance rating not less than one and one-half (1 1/2) hours and shall comply with the requirements of clause 3.517.17 for stair exitway doors. The door from the vestibule to the stairway shall be a tight-fitting door, equal to not less than an exterior type solid wood door without voids, asembled with exterior glue, thirty-two millimetres minimum thickness set in a steel frame. Wired glass, if provided, shall not exceed 654 sq. centimetres (.0654 m^2) in area and shall be set in a steel frame. The door shall be provided with a drop sill and be sealed at all edges to minimize air leakage.

3.519.5 TERMINAL PASSAGEWAY - The smoke-proof enclosure shall terminate at ground level and shall provide egress to the street independently of all other exitways. When ground floor passageways are used, they shall comply with the requirements of subsection 3.512 except that there shall not be openings therein other than the smoke-proof enclosure and street exit doorways. The passageway walls should be of four (4) hour fire-resistance rated construction, and the floor and roof of three (3) hour fire-resistance rated construction.

3.519.6 CONSTRUCTION - The construction of smoke-proof enclosures shall be of walls with a four (4) hour fire-resistance rating without openings other than the required doorways. The vestibule shall be considered to be an element of the exitway and shall be constructed in accordance with the fire-resistance rating requirements in Part 3 Section 1 and Table 3.114. The balcony shall be constructed in accordance with the fire-resistance rating requirements in Table 3.114 for floor construction. The stairshaft vestibule or balcony shall be provided with emergency lighting from an approved independent power source to assure continued illumination in case of emergency.

3.519.7 VENTILATION OF SMOKE-PROOF ENCLOSURES - Smoke-proof enclosures/fire escape towers shall be ventilated, preferably with natural ventilation, but if mechanical ventilation is used, it shall comply with clause 3.519.9 below.

3.519.8 NATURAL VENTILATION - The balcony separating the smoke-proof enclosure from the interior building spaces shall have at least one (1) open side adjacent to a street, alley, or yard with guard railings across the open side(1). One (1) open side of the balcony shall have a minimum open area of one and one half square metres (1.5 m^2). The balcony floor shall be level with or installed below the building floor but a step shall be permitted between the balcony and the smoke-proof enclosure. The street, alley, or yard adjacent to one (1) open side of the balcony shall have a minimum area of twenty sq. metres, (20 m^2) and a minimum dimension of 3 metres.

3.519.9 MECHANICA VENTILATION - If mechanical ventilation is to be provides, it must comply with the following:

(a) the mechanical ventilation system shall be automatically activated on at least every third floor in the height of the tower;

(b) it must not be dependent on the normal power supply in the building;

(c) the system both in the tower and in vestibules on each floor if they are closed-in and not open-sided as required under 3.519.8 above shall have the capacity to give a complete air change in all areas every minute

(d) for closed-in type vestibules, the ceiling shall be at least .5 metres above the height of doors to form a smoke trap, the incoming air shall be within 150 millimetres of the floor and the exhaust registers within 150 mm of the ceiling. The exhaust pressure shall be 150 percent of the supply pressure.

3.519.10 STAIRTOWER VENTILATION (MECHANICAL) - Supply air shall be introduced at the base of the tower adjacent to exitway discharge, and the supply shall be sufficient to build up a small pressure above normal atmospheric pressure with all doors closed. Discharge shall be at the top of the shaft.

3.519.11 EMERGENCY LIGHTING - All vestibules, whether open-sided or closed-in and the stair shaft/tower shall be provided with emergency lighting. The standby generator which is installed for the vestibule and stairshaft mechanical ventilation equipment may be used for the standby emergency lighting power supply, or if tower is ventilated by natural means then battery operated emergency lighting shall be installed. As recommended elsewhere in this Code, a set of batteries in a suitable cupboard with a trickle-charger from mains electricity is an acceptable power supply for emergency lighting.

3.519.12 FIRE PROTECTION INDICATOR PANEL - A fire protection indicator panel may be required by the Fire Department and, if so, shall be located as near as practicable inside the entrance to the smoke-proof tower stairshaft at ground level. This panel shall indicate the floor or floors having caused the alarm. The panel shall have an overriding manual switch capable of deactivating the ventilation equipment if installed.

3.519.13 FIRE DEPARTMENT COMMUNICATIONS CONNECTION - The fire protection indicator panel shall have a direct connection to the fire department facilities if required by the Building Authority and/or the Fire Department.

3.519.14 ACCEPTANCE AND TESTING - Before any of the foregoing equipment is accepted by the Building Authority, it shall be tested in the presence of an officer of the Fire Department to confirm that equipment is operating in compliance with these requirements.

3.519.15 BUILDING OWNER'S RESPONSIBILITY - All equipment referred to in the above clauses shall be tested at least once every month by the Fire Authority or by some approved reputable firm acceptable both to the owner, the Local Authority and/or the Fire Board. A log of tests made shall be maintained and shall be available for inspection at any time by all interested parties.

3.520 Exterior Exitway Stairways

3.520.1 AS REQUIRED EXITWAYS - Exterior stairways conforming to the requirements of interior stairways in all respects, except as to enclosures and except as herein specifically modified, may be accepted as an element of a required means of egress in buildings not exceeding five (5) stories or twenty metres (65 feet) in height for other than occupation group I (institutional) buildings except as provided in clause 3.520.2 for residential buildings. Exterior stairways which are accepted as exitway elements shall be relieved from requirements for fire doors, but shall be provided with handrails and guards as required for interior exitway stairs.

3.520.2 LOCATION AND ARRANGEMENT - Exterior stairways may be utilized where at least one (1) door from each tenant opens onto a roofed-over open porch or balcony served by at least two (2) stairways, except that one (1) stairway may be provided as permitted in Table 3.511, and so located as to provide a choice of independent, unobstructed means of egress directly to the ground. Such porches and stairways shall comply with the requirements for interior exitway stairways as specified in sub-section 3.517. Porches and balconies shall be not less than 1.4

metres in width. The stairways shall be located far apart from each other. The maximum travel distance from any tenant space to the nearest stairway shall be as specified in Table 3.509. Porches and stairways shall be located at least three metres (3.0 m) from adjacent property lot lines and from other buildings on the same lot, unless openings in such buildings are protected by three-quarter (3/4) hour fire-resistance rated doors or windows.

3.520.3 GUARDS AND HANDRAILS - Guards and/or handrails shall be as required in sub-section 3.517.

3.520.4 OPENING PROTECTIVES - Openings below and within three metres (3.0 m) horizontally of the stairways shall be protected with approved three-quarter (3/4) hour fire-resistance rated automatic opening protectives.

3.520.5 ACCESS TO STREET - All required exterior stairways shall be located so as to lead directly to a street or open space with direct access to a street or when located on the rear of the building may lead through a passageway at ground level complying with Section 3.512.

3.520.6 PROJECTION - Exterior stairways shall not project beyond the street lot line.

3.520.7 CONSTRUCTION - Exterior stairs, porches and balconies shall be constructed of materials consistent with the types of materials permitted in Part 3 Section 1 (Table 3.114) of this Code for the type of construction of the building to which the stairway is attached.

3.521 Moving Exitway Stairways

3.521.1 WHEN ACCEPTABLE - Moving stairways of the horizontal non-slip tread type moving in the direction of egress may be accepted as an approved exitway element in buildings of all occupancy groups except assembly (A) and institutional (I) occupancies, when constructed and approved in accordance with the requirements of sub-section 3.517 of this Code.

3.521.2 OTHER REQUIREMENTS -

(a) if/when the power supply fails they can be used as a normal stairway in all respects.

(b) they must be enclosed in fire-resistance rated partitions as is called for under 3.517.

(c) they must not have a longer run of vertical travel than two stories in height or more than 8 metres; and

(d) they must be completely constructed in non-combustible materials.

3.522 **Fire Escapes**

5.522.1 WHERE PERMITTED - Fire escapes shall only be permitted as an element of a required means of egress on existing buildings or structures when constructed in accordance with the approved rules and when more adequate exitway facilities cannot be provided. Fire escapes shall not provide more than fifty (50) percent of the required exit capacity.

3.522.2 LOCATION - When located on the front of the building and projecting beyond the building line, the lowest landing shall be not less than 2.4 metres or more than 3.6 metres above ground level, and shall be, equipped with a counter-balanced stairway to the street. In alleyways and thoroughfares less than 10 metres wide, the clearance under the lowest landing shall be not less than 3.6 metres.

3.522.3 CONSTRUCTION - The fire escape shall be designed to support a live load of 488 kg per square metre (kg/m^2) and shall be constructed of steel or other approved non-combustible materials with the approval of the Local Authority/Local Fire Board, imported hardwood that does not burn, or "Pyrolith", or other approved fire-proofed timber may be used for decking of fire escapes and treads and handrails only of escape stairs provided it is not less than 50 millimetres thick. Fire escapes may be constructed of wood not less than 50 millimetres thick on buildings of Type 4 construction.

3.522.4 DIMENSIONS -

(a) Escape walkways and stairways shall have a minimum clear width of 560 millimetres.

(b) Stairs shall have risers of not more than, and treads of not less than 203 millimetres.

(c) Landings at foot of stairs shall not be less than 1.0 metre wide and 1.0 metre long.

(d) handrails to walkways and stairways shall be not less than 760 millimetres not more than 900 millimetres high.

3.522.5 OPENING PROTECTIVES - All doors, windows, vents, exhausts and grilles along the line of exitways or stairs that are not actual exit openings from rooms shall have approved hinged protective shutter/screens of one-half hour fire-resistance rating.

3.523 Exit Signs (and Lighting for)

3.523.1 LOCATION - In all buildings having an occupancy load of fifty (50) or more all required means of egress shall be indicated with approved internally illuminated signs reading "Exit" visible from the exitway access and, when necessary, supplemented by directional signs in the access corridors indicating the direction and way of egress. All exit signs shall be located at exitway doors or exitway access areas, so as to be readily visible.

3.523.2 SIZE AND COLOUR - Exit signs shall have green letters at least 150 millimetres high and the minimum width of each stroke shall be twenty millimetres on a white background or in other approved distinguishable colours. If an arrow is proved as part of an exit sign, the construction shall be such that the arrow direction cannot be readily changed. The letters "Exit" shall be clearly discernible when the internally illuminated sign is not energized. (See also below under 3.524 for required lighting).

3.523.3 ILLUMINATION

3.523.4 In all buildings that are occupied only during daylight hours such as most schools and office buildings, certain factories, storage buildings and others where the Local Authority/Fire Authority consider that there is sufficient natural light during working hours for occupants to read the fire exit signs, illumination of such signs is not mandatory.

3.523.5 In buildings that are occupied at nights as well as daytime and in buildings where there is insufficient or no natural light during daylight hours, all exit signs shall be illuminated from an approved source. (See under 3.524 for requirements for exitway and escape lighting).

3.524 Means of Egress Lighting

3.524.1 ARTIFICIAL LIGHTING - All means of egress in other than one and two-family dwellings shall be equipped with artificial lighting facilities to provide the intensity of illumination herein prescribed continuously during the time that conditions of occupancy of the building require that the exitways be available. Lighting shall also be provided to illuminate the exitway discharge. As set out in 3.523.4 above for lighting of signs the Local Authority/Fire Authority can waive the necessity for illumination of both signs and exitways from certain buildings.

3.524.2 INTENSITY OF ILLUMINATION - The horizontal illuminance on the centre line of any escape route should never be less than 0.2 lux, and this level of emergency lighting shall be provided immediately, or at most shall be provided within five seconds of failure of the normal lighting. This calls for automatic change over to batteries or other emergency supply in places of assembly and other public buildings.

3.524.3 MOTION PICTURE THEATRES AND OTHER AUDITORIUMS - In places of assembly for the exhibition of motion pictures or other projections by means of directed light, the illumination where the recommended MAINTAINED level of illuminance of 0.2 lux is likely to effect normal working, it is acceptable to reduce this level to not less than 0.02 lux, provided that the system is so arranged that in the event of the failure of the normal system of lighting within the auditoriums, the level of escape lighting illuminance shall be immediately and automatically restored to a minimum of 0.2 lux.

3.524.4 EMERGENCY LIGHTING SYSTEM - Means of egress lighting shall be provided from an independent power source or other approved auxiliary source of power to assure continued illumination in case of emergency or primary power loss for a duration of one (1) hour in the following:

(a) Occ. group A (public assembly)
(b) Occ. group B (business) containing more than one thousand (1,000) occupants.
(c) Occ. group I (institutional)
(d) Occ. group M (mercantile) when greater than three hundred (300) square metres in area on any floor or when having one (1) or more floors above or below ground floor.
(e) Occ. group R-1 (hotels) containing more than twentyfive (25) sleeping rooms;
(f) Occ. group R-2 (multi-family dwellings) containing more than fifty (50) occupants; and
(g) in all windowless buildings or portions thereof regardless of Occ. group, except R-3.

NOTE: For more detailed requirements, it is recommended that Code of practice for The Emergency Lighting of Premises, BS 5266 parts 1 and 2 (1975) be consulted and used, especially those sections dealing with:

(a) Escape lighting
(b) Standby lighting
(c) Maintained emergency lighting and
(d) Non-maintained emergency lighting

3.525 **Hazards to Means of Egress**

3.525.1 FLOOR OPENINGS - Manholes or floor access panels shall not be located in the line of egress which reduce the clearance to less than 810 millimetres.

3.525.2 PROTRUSIONS - There shall not be low-hanging door closers that remain within the opening of a doorway when the door is open or that protrude hazardously into corridors or line of egress when the door is closed. There shall not be low-hanging signs, ceiling light or similar fixtures which protrude into corridors or lines of egress.

3.525.3 IDENTIFICATION OF HAZARDOUS EXITS - Doors leading to dangerous areas such as fire escapes, loading platforms, switch rooms and mechanical rooms shall be equipped with knobs, handles or push bars that have been knurled.

3.525.4 FLOOR SURFACES - All floors of corridors and lines of egress shall have a surface that is non-slip.

CARIBBEAN UNIFORM BUILDING CODE

PART 3
OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 6

FIRE RESISTIVE CONSTRUCTION REQUIREMENTS

PART 3

SECTION 6

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PART 3

SECTION 6

3.600 FIRE-RESISTIVE CONSTRUCTION REQUIREMENTS

3.601 Scope

3.601.1 The provision of this Section (6) of the Building Code shall govern the use and design of all materials and methods of construction in respect to required fire-resistance rating and flame-resistance as determined by the potential fire hazard of the use and occupancy of the building or structure and the location and function of all integral structural and other fire-protective elements of the building; and the installation of safeguards against the spread of fire to and from adjoining structures.

3.602 General Requirements

3.602.1 PERFORMANCE STANDARDS - The requirements of this Section shall constitute the minimum functional performance standards for fire protection purposes; and shall not be deemed to decrease or waive any strength provisions or in any other manner decrease the requirements of this Code in respect to structural safety.

3.602.2 USE OF COMBUSTIBLES - All materials and forms of construction that develop the fire-resistance rating required by this Code shall be acceptable for fireproofing and structural purposes; except that the use of combustible component materials in structural units or structural assemblies shall be limited in types of construction specified in Part 3 Section 1 (3.114 to 3.118.1 and including Table 3.114) and in sub-section 3.604 below.

3.602.3 COMBUSTIBLE COMPONENTS - Combustible aggregates may be incorporated in concrete mixtures approved for fire-resistance rated construction, for example gypsum concrete, cinder concrete, and other approved component material or admixture used in assemblies that meet the fire-resistance test requirements of this Code; and wood nailing strips or any other material of similar combustible characteristics may be embedded in concrete and masonry construction for securing trim and finish.

3.603 Plans and Specifications

3.603.1 All plans prepared and submitted for approval for all buildings shall designate the type of construction (types 1, 2, 3, or 4) and the fire-resistance rating of all structural elements as required by this code. The plans and/or specifications, if so required by the Local Authority shall include documentation or supporting data substantiating all required fire-resistance rating.

3.604 Fire Hazard Classification

3.604.1 It is imperative that these clauses 3.604.1 to 3.604.3 and accompanying Table 3.604 below, be read in conjunction with Part 3 Section 1 of this Code, and particularly clauses 3.114.1 to and including 3.118.1 and Table 3.114 therein.

3.604.2 GENERAL - The degree of fire hazard of buildings and structures for each specific occupancy group as defined by the fire grading in Table 3.604 shall determine the requirements for fire walls, fire separation walls and the segregation of mixed uses as prescribed in Section 1, clauses 3.114.1 to 3.118.1 and all structural members supporting such elements unless otherwise provided for in this Code.

3.604.3 UNCLASSIFIED USES/OCCUPANCIES - The Local Authority shall determine the fire hazard classification of a building or structure design for uses not specifically provided in Table 3.604 in accordance with the fire characteristics and potential fire hazard of the occupancy group which it most nearly resembles; or its designation shall be fixed by the approved rules.

3.605 Fire-Resistance Tests

3.605.1 STRUCTURAL BUILDING ASSEMBLIES - Built-up masonry units and composite assemblies of structural materials including walls, partitions, columns, girders, beams and slabs, assemblies of slabs and beams or other combinations of structural units for use in floor and roof construction shall be regulated by the fire-resistance ratings of Table 3.114 in Part 3 Section 1 of this Code. The fire-resistance ratings of the floor and ceiling assemblies shall extend to and be tight against the exterior wall.

3.605.2 TESTS WITHOUT LOAD - (COLUMNS, BEAMS AND GIRDERS) -To evaluate column, beam and girder protection for structural units when the fireproofing is not a structural part of the element, in lieu of full size tests of loaded specimens, the structural sections encased in the material proposed for use as insulation and fire protection may be subjected to the standard test procedure without load.

TABLE 3.604

FIRE GRADING OF OCCUPANCY GROUPS

Class	Occupancy Group	Fire Grading In Hours
A-1	Assembly theatres	3
A-2	Assembly, night clubs	3
A-3	Assembly, recreation centres, lecture halls, terminals, restaurants	2
A-4	Assembly, churches, schools	1 1/2
B	Business	2
F	Factory and industrial	3
H	High hazard	4
I-1	Institutional, restrained occupants	3
I-2	Institutional, incapacitated occupants	2
M	Mercantile	3
R-1	Residential, hotels	2
R-2	Residential, multi-family dwellings	1 1/2
R-3	Residential, 1 and 2-family dwellings	1
S-1	Storage, moderate hazard	3
S-2	Storage, low hazard	2

3.605.3 ALTERNATE PROTECTION - When it can be shown to the Local Building Authority that the structural integrity of structural framing elements will not be reduced below a safe level by a fire (within the building or in an adjacent building) having a severity corresponding to the fire-resistance rating required for the elements, through the use of heat shields, separations or other approved means of protection, fire-protective coverings or insulating enclosing materials need not be provided for such elements.

3.605.4 ROOF COVERING: TEST PROCEDURE AND CLASSIFICATION - Roof covering materials shall be classified in accordance with the severity of exposure to exterior fire and ability to resist the spread of fire from surrounding buildings when tested in accordance with roof covering standard such as ASTM E108 - 175 or equivalent British or Canadian Standard.

3.605.5 CLASS A ROOFINGS - Are those which are effective against severe fire exposure. In addition to roof coverings which have been classified, asbestos cement, metal, portland cement concrete, slate, concrete masonry and tile are acceptable where Class A roof coverings are required.

3.605.6 Class B ROOFINGS - Are those which are effective against moderate fire exposure.

3.605.7 CLASS C ROOFINGS - Are those which are effective against light fire exposure.

3.605.8 NON-CLASSIFIED ROOFINGS - Are those not tested.

3.605.9 OPENING PROTECTION - (See also under Means of Egress Part 3 Section 5 of this Code) - Opening protectives shall include fire doors, fire windows, or fire dampers, and all required hardware, anchorage, frames and sills necessary for such assemblies.

3.605.10 LABELLED (CERTIFIED) FIRE DOORS - Opening protective door assemblies including the frames, hardware and operation which comply with a standard such as ASTM or BSI and accepted practice, including shop inspection, by an accredited testing or inspection agency shall be deemed to meet the requirements of this Code for their recommended and approved locations and use as listed in sub-section 3.617 below (Fire Doors).

3.605.11 LABELLED FIRE WINDOWS AND SHUTTERS - Fire window and shutter assemblies which comply with clauses under 3.618 (Fire Windows and Shutters) and relevant ASTM/BSI standards, and accepted practice of an accredited testing or inspection agency, shall be deemed to meet the requirements of this Code and the required locations of such assemblies.

3.605.12 COMBUSTIBILITY TESTS - Where the behaviour of materials under exposure to fire tests is specified in this Code, the characteristics of materials shall be determined by the following tests and criteria.

3.605.13 TESTS - The following tests shall serve as criteria for acceptance of building materials (when tested in the form and thickness in which they are used) as set forth in sub-sections 3.115, 3.116 and 3.117 of the Code, governing the combustibility of building materials for use in Types 1, 2 and 3 construction.

(a) Materials which pass the test procedure for defining non-combustibility of elementary materials set forth in ASTM E 136 (or equivalent BSI Standard) when exposed to a furnace temperature of seven hundred and fifty (750) degrees C for a period of five (5) minutes, and do not cause a twelve (12) degrees C above the furnace air temperature at the beginning of the test and which do not flame after an exposure of thirty (30) seconds.

(b) Materials having a structural base of non-combustible material as defined in paragraph (a) above, with a surfacing not more than three millimetres (3) thick which has a flame-spread rating not greater than fifty (50) when tested in accordance with the method of test for surface burning characteristics of building materials as set forth in ASTM E S4 or equivalent standard.

Note: The term "non-combustible" does not apply to the flame-spread characteristics of interior finish or trim materials. A material shall not be classed as a non-combustible building construction material which is subject to increase in combustible or flame-spread rating beyond the limits herein established through the effects of age, moisture, or other atmospheric conditions.

3.605.14 TESTS FOR FIRE-RETARDANT-TREATED TIMBER - Where permitted for use as a structural element, fire-retardant-treated wood shall be tested in accordance with the standard method of test for surface burning characteristics of building materials (tunnel test) and shall show a flame-spread rating not greater than twenty-five (25) when exposed for a spread of not less than thirty (30) minutes, without evidence of significant progressive combustion. The material shall bear the identification of an accredited authoritative testing or inspection agency, showing the performance rating thereof. As above, all tests shall be carried out in accordance with the ASTM and/or BSI relevant standards for surface spread of flame.

3.605.15 USE LIMITATIONS - Timber that has been pressure treated with fire-retardant chemicals in accordance with the standards for pressure treatment of lumber or plywood in buildings or treated by other approved means during manufacture may be used in Types 1 and 2 construction for partitions, structural elements and roof framing and sheathing as indicated by Note h in Table 3.114 (Part 2 Section 1 of this Code), provided that the assembly in which such material is used shall produce the required fire-resistance rating when tested in accordance with the standard method of fire test for building construction and materials. Where the material is to be subjected to sustained high humidity or exposed to the weather, it shall be further identified to indicate that there is not an increase in listed fire hazard classification after being subjected to the Underwriters' Laboratories (UL) Standard Rain Test. Where used as a structural element, such material shall meet the requirements of clauses 3.605.14 and 3.605.15 above. Where used as interior finish, such material shall meet the requirements of sub-section 3.606 below.

3.606 Flame-Resistance Tests

3.606.1 GENERAL - All materials which are required to restrict the spread of flame or to be flame-resistant under the provisions of this Code, including, but not limited to, interior finish materials, fire-retardant-treated timber, tents and tarpaulins, and interior hangings and decorations, shall meet the requirements for their respective use and classifications as determined by the applicable test procedures of ASTM or BSI Standards.

3.606.2 INTERIOR FINISH MATERIALS - All materials used for interior finish shall be classified in accordance with the Method of Test for Surface Burning Characteristics of Building Materials as per relevant standards given above.

3.606.3 INTERIOR HANGINGS OR DECORATIONS - Where required to be flame-resistant under the provisions of this code all materials specified or required for artistic enhancement or use for decorations, draperies, curtains, scenery and hangings shall comply with this section for non-combustible or fire-retardant materials or if treated to be flame-resistant shall not generate smoke or gases more dense or more toxic than those given off by untreated wood or paper burning under comparable conditions when tested in the vertical flame test. (Relevant Standards as above).

TABLE 3.606

INTERIOR FINISH CLASSIFICATION

Class of Material	Surface burning characteristics test (Tunnel Test)
1	0 to 25
11	26 to 75
111	76 to 200

3.606.4 LIMITATION OF APPROVAL - All approvals of organic decorative material shall be limited to one (1) year. The owner or his authorized agent shall file an affidavit with the Building Authority certifying that the process and materials used, comply with this Code and stating the date of treatment and the warranted period of effectiveness of the process.

3.606.5 FIELD TEST FOR DECORATIVE MATERIALS - An inspector for the Local Authority (preferably accompanied by an officer of the Local Fire Authority) shall subject decorative materials where required to be flame-resistant, to a field test consisting of the application of the flame from a standard 19 mm paraffin candle for a period of one (1) minute.

3.606.6 REPLACEMENT OF DEFECTIVE MATERIALS - All treated hangings, draperies, canvas and other decorative and tent materials that fail to meet the field test requirements shall be re-treated or replaced by an approved installation.

3.607 Special Fire-Resistive Requirements

3.607.1 GENERAL - In buildings or parts thereof of the uses and types of construction herein specified, the general fire-resistive requirements of Tables 3.114 (Part 3 Section 1 of Code) and the height and area limitations of Table 3.205 (Part 3 Section 2 of this Code) shall be subject to the exceptions and modifications described in the following clauses 3.607.2 to 3.607.5.

3.607.2 PUBLIC GARAGES - All existing buildings and structures altered or converted for use to a garage, motor vehicle repair shop or gasoline service station, more than one (1) storey in height, unless of fireproof (Type 1) construction, or heavy timber Type 3-A) construction, shall have the partitions, columns and girders and all floor and roof construction protected and insulated with non-combustible materials or assemblies of component materials having a fire-resistance rating of not less than one (1) hour; except that existing roof trusses shall be exempt from all fireproofing requirements.

3.607.3 PETROLEUM BULK STORAGE BUILDINGS - Warehouses for the bulk storage of not more than two hundred thousand litres (200,000 l) of lubricating oils with a flash point of not less than one hundred and fifty degrees Celsius (150^{o}C) in approved sealed containers may be erected outside the fire limits, of masonry wall (Type 3) construction, not more than four hundred and fifty square metres (450 m^2) in area and not more than one (1) storey or six metres (6 m) in height, or to proportionate areas in other types of construction as regulated by Table 3.205 in Part 3 Section 2 of this Code. Not more than one (1) motor vehicle may be stored in such buildings unless separately enclosed with a fire separation wall of two (2) hour fire-resistance rating.

3.607.4 PACKING AND SHIPPING ROOMS - Every packing or shipping room located on or below a floor occupied for occupancy group M (mercantile) use shall be separated therefrom by fire separation walls or floor-ceiling assemblies of not less than the fire-resistance rating of the type of construction but not less than one (1) hour fire-resistance rating.

3.607.5 TRUCK LOADING AND SHIPPING AREAS - Truck loading and shipping areas shall be permitted within any occupancy group B (business) building, provided such areas are enclosed in construction of not less than the fire-resistance rating of the type of construction as set forth in Table 3.114, Part 3 Section 1 of this Code, but not less than one (1) hour, and direct access is provided therefrom to the street.

3.607.6 OCCUPANCY GROUP R (RESIDENTIAL) BUILDINGS

3.607.7 PROTECTED ORDINARY CONSTRUCTION - Multi-family dwellings (occupancy group R-2) of protected ordinary (Type 3-B) construction may be increased to six (6) stories or twenty-three metres (23 m) in height when the first floor construction above the basement or cellar has a fire-resistance rating of not less than three (3) hours and the floor area is subdivided by two (2) hour fire walls into fire areas of not more than two hundred and eighty square metres (280 m^2).

3.607.8 PROTECTED NON-COMBUSTIBLE CONSTRUCTION - When of protected non-combustible (Type 2-B) construction, multi-family dwellings (occupancy group R-2) may be increased to nine (9) stories or thirty metres (30 m) in height when separated by not less than fifteen metres from any other building on the lot and from interior lot lines, the exitways are segregated in a fire area enclosed in a fire wall of two (2) hour fire-resistance rating and the ground floor construction has a fire-resistance rating of not less than one and one-half (1 1/2) hours.

3.607.9 RETAIL BUSINESS USE - The ground floor of buildings of unprotected non-combustible (Type 2-C), masonry wall (Type 3-C) or frame (Type 4-B) construction may be occupied for retail store use, provided the floor-ceiling assembly and enclosure walls are protected to afford one (1) hour fire-resistance rating and the exitways from the residential floors are separately enclosed in accordance with the requirements of Part 3 Section 5 (Means of Egress) of this Code.

3.607.10 GROUND FLOOR PROTECTION

3.607.11 NON-FIREPROOF CONSTRUCTION - In all buildings other than one and two-family dwellings (occupancy group R-3) and other than fireproof (Type 1) construction with habitable or occupiable stories or basements below ground level, the floor-ceiling assemblies and supports below the ground floor shall be protected by one (1) of the following methods.

(a) fire-resistance rating of not less than 1 hour; or

(b) heavy mill (Type 3-A) construction; or

(c) an automatic fire suppression system.

3.607.12 PROTECTED NON-COMBUSTIBLE CONSTRUCTION - In all buildings of protected non-combustible (Type 2-A) construction, more than four (4) stories or fifteen metres (15 m) in height, in other than residential (R) occupancy groups, the floor-ceiling assembly above the basement of cellar shall be constructed with a fire-resistance rating of not less than two (2) hours.

3.607.13 BASEMENT ASSEMBLY USES - Places of public assembly for amusement, entertainment, instruction, or service of food or refreshment shall not be located in stories or rooms below ground unless the floor-ceiling assembly above and below is of not less than one and one-half (1 1/2) hour fire-resistance rating.

3.607.14 NON-COMBUSTIBLE CONSTRUCTION EXEMPTIONS - One (1) storey buildings of Type 2-C construction which do not exceed two hundred and eighty square metres (280 m^2) in area in all occupancy groups except high hazard (H), assembly (A) and institutional (I) shall be exempt from all protected exterior wall requirements.

3.607.15 INTERIOR PARTITIONS - In buildings of other than Institutional (I) and Residential (R) occupancy groups of fireproof (Type 1) and protected non-combustible (Type 2-A and 2-B) constructions, partitions of a single thickness of wood or approved composite panels, and glass or other approved materials of similar combustible characteristics, may be used to subdivide rooms or spaces into offices, entries, or other similar compartments, provided they do not establish a corridor serving an occupant load of thirty (30) or more (in areas occupied by a single tenant) and not exceeding four hundred and sixty square metres (460 m^2) may be subdivided with fire-retardant-treated wood when complying with clauses 3.605.14 and 3.605.15.

3.607.16 EXHAUST AIR, AND AIR DUCTS (FOR AIR-CONDITIONED ROOMS) - The use of uninhabited basements, cellars, crawl spaces, cavity walls, areas above ceilings or attic spaces as supply, make up, exhaust air or return air plenums or ducts is prohibited.

3.607.17 FIRE DAMPERS TO DUCTED AIR - Except when proper fire tests have shown that fire dampers are not necessary to maintain the integrity of the fire-resistance rated assembly, fire dampers complying with the relevant standards shall be installed in the following locations:

(a) Ducts penetrating a fire wall. (When a fire wall is of three (3) hour or greater fire endurance, a fire door is required);

(b) Ducts passing through a fire separation wall.

(c) Ducts penetrating a fire-resistance rated shaft wall. Sub-ducts extending 600 millimetres vertically upward may be used in lieu of fire dampers for exhaust ducts.

(d) Ducts penetrating the ceiling of a fire-resistance rated floor/roof-ceiling assembly.

(e) Ducts penetrating fire-resistance rated corridor walls, unless the building is completely sprinklered or unless the ducts are part of an engineered smoke removal system.

3.608 Exterior Walls

3.608.1 GENERAL - All exterior walls should comply with the structural provisions of Part 2 (Structural Design Requirements) of this Code and with the fire-resistance rating requirements of Table 3.114 (Part 3 Section 1 of this Code).

3.608.2 EXCEPTIONS - The provisions of this Code shall not be deemed to prohibit the omission of exterior walls for all or part of a storey when required for special uses and occupancies; except that when so omitted, the open areas shall be separated from the rest of the area and from the upper and lower stories of the building by wall and floor construction of the fire-resistance rating required in Table 3.114 as above, and except as otherwise specifically permitted in this Code, the piers, columns and other structural supports within the open portion shall be constructed with the fire-resistance rating required for exterior bearing walls in Table 3.114 (Part 3 Section 1 of this Code).

3.608.3 VERTICAL SEPARATION OF WINDOWS - Where required in all buildings designed for business (B) factory and industrial (F), high hazard (H), Mercantile (M) or storage (S) occupancies, exceeding three (3) stories or thirteen metres (13 m) in height, openings located vertically above one another in exterior walls which are required to have a fire-resistance rating of more than one (1) hour shall be separated by apron or spandrel walls not less than one metre (1.0 m) in height extending between the top of any opening and the bottom of the opening next above.

3.608.4 FIRE RESISTANCE RATING - These apron or spandrel walls as above shall be constructed with the same fire-resistance rating required for the exterior wall in which it is located as specified in Table 3.114, except when such required rating exceeds one (1) hour, approved wire glass construction in fixed non-combustible sash and frames not exceeding one-third (1/3) of the area of such apron or spandrel may be located therein, and except further that in exterior non-bearing enclosure walls which are not required to be more than one (1) hour fire-resistance rating the provisions of this section in respect to apron or spandrel walls shall not apply.

3.609 Fire Walls and Party Walls

3.609.1 GENERAL - Walls shall have sufficient structural stability under fire conditions to allow collapse of construction on either side without collapse of the wall and shall be constructed of any approved non-combustible materials providing the required strength and fire-resistance rating specified in Table 3.114 for the type of construction, but not less than the fire grading of the occupancy group specified in Table 3.604 in this section of the Code. The construction shall comply with all the structural provisions for bearing or non-bearing walls of this Code.

3.609.2 SOLID MASONRY - When constructed of solid masonry, the wall thickness shall be not less than the requirements of Part 2 Section 4 of this Code.

3.609.3 REINFORCED CONCRETE - When constructed of reinforced concrete, the wall thickness shall be not less than two hundred millimetres (200 mm) for the uppermost ten metres (10 m) or portion thereof measured down from the top of the wall.

3.609.4 CUTTING WALLS - A wall 200 mm or less in thickness, shall not be cut for chases or socketed for insertion of structural members subsequent to erection.

3.609.5 HOLLOW WALLS - When combustible members frame into hollow walls or walls of hollow unit, all hollow spaces shall be solidly filled for the full thickness of the wall and for a distance not less than 100 mm above, below and between the structural members, with non-combustible materials approved for firestopping, (see 3.616 below). The wall shall be not less than the minimum thickness specified in the Building Code Requirements for Masonry as indicated in Part 2 Section 4 of this Code.

3.609.6 COMBUSTIBLE INSULATION - The Building Authority may permit the application of cork, fibreboard, or other combustible insulation if laid up without intervening air spaces and attached directly to the face of the wall, and protected on the exposed surface as required by this Code.

3.609.7 CONTINUITY OF WALLS - In all buildings, the walls shall be continuous from foundation to eight hundred millimetres (800 mm or expressed in metres 0.8 m) above the roof of the building except for the following:

(a) The wall may terminate at the underside of the roof deck where the roof is of non-combustible construction and is properly firestopped at the wall.

(b) The wall may terminate at the underside of the roof deck in Types 3 and 4 construction if properly firestopped, and the roof sheathing or deck is constructed of approved non-combustible materials for a distance of one and one-quarter metres (1.25 m) on either side of the wall and combustible material does not extend through or over the wall.

3.609.8 OFFSET FIRE WALLS - If fire walls are offset at intermediate floor levels in fire-protected skeleton frame construction, the offset floor construction and the intermediate wall supports shall be constructed of non-combustible materials with a fire-resistance rating not less than that required for the fire wall.

3.610 Fire Wall Openings

3.610.1 GENERAL - Opening in fire walls shall not exceed the limits in size and area herein prescribed and the opening protectives shall conform to the provisions of 3.605 and 3.616 in this section of the Code.

3.610.2 SIZE OF OPENING - Except in sprinklered buildings, an opening through a fire wall shall not exceed eleven square metres (11 m^2) in area and aggregate width of all openings at any floor level shall not exceed twenty-five (25) percent of the length of the wall.

3.610.3 EXCEPTION - GROUND FLOOR ONLY - When the entire areas on both sides of a fire wall are protected with an approved automatic fire suppression system complying with the requirements of Part 3 Section 7 of this Code, openings designed for the passage of trucks may be constructed not more than twenty-two and one-half square metres (22.5 m^2) in area with a minimum distance of one metre (1.0 m) between adjoining openings. Such openings shall be protected with approved automatic protectives of three (3) hour fire-resistance rating and/or provided with an approved water curtain for such openings in addition to all other requirements.

3.610.4 OPENING PROTECTIVES - Every opening in a fire wall shall be protected (on both sides, if required by the Local Authority/Fire Board) with an approved automatic protective assembly as herein required, or the approved labelled equivalent, except horizontal exit openings.

3.610.5 HOLD-OPEN DEVICES - Heat-actuated hold-open devices used on an automatic fire assembly providing three (3) hour fire-resistance rating shall be installed, one (1) on each side of the wall at ceiling height where the ceiling is more than one metre (1.0 m) above the opening. Fire assemblies protecting openings required to have one and one-half (1 1/2), one (1) or three-fourths (3/4) hour fire-resistance rating, and which are not exitway doors, may be activated in a similar manner, or by a single fusible link incorporated in the closing device. Doors opening in a means of egress route shall be closed by actuation of a smoke detector.

3.611 Fire Separation Walls

3.611.1 MIXED OCCUPANCIES - When a building contains more than one (1) occupancy, and each part of the building is separately classified as to use, the mixed occupancies shall be completely separated with fire separation walls as specified in Part 3 Section 1, Clauses 3.113.1 - 3.113.3 of this Code.

3.611.2 ONE AND TWO-FAMILY DWELLNGS - The requirements for the construction of fire separation walls in buildings containing single-family dwellings or two-family dwellings (occupancy group R-3) are as below 3.611.3 - 3.611.6.

3.611.3 TWO-FAMILY DWELLING - SUPERIMPOSED DWELLING UNITS - When one (1) dwelling unit of a two-family dwelling is located wholly or partly above the other dwelling unit, the two (2) dwelling units shall be completely separated by fire separation rating walls and floor-ceiling assemblies of not less than one (1) hour fire-resistance rated construction.

3.611.4 TWO-FAMILY DWELLING, SIDE-BY-SIDE DWELLING UNITS - When adjacent dwelling units of a two-family dwelling are attached by a common wall, this wall shall be a fire separation wall, having a minimum one (1) hour fire-resistance rating that shall serve to completely separate the dwelling units.

3.611.5 MULTIPLE, SINGLE-FAMILY DWELLING - SIDE-BY-SIDE - When multiple, single-family dwellings (occupancy group R-3) are attached by a common wall, such wall shall be a fire separation wall, having a minimum one (1) hour fire-resistance rating. Wall shall extend from the foundation to the underside of the roof sheathing, and to the inside of the exterior wall sheathing.

3.611.6 MULTIPLE, TWO-FAMILY DWELLINGS - SIDE-BY-SIDE - When multiple, two-family dwellings (occupancy group R-3) are attached by a common wall, this wall shall be a fire separation wall, having a minimum one (1) hour fire-resistance rating, and the wall shall extend from the foundation to the underside of the roof sheathing and to the inside of the exterior wall sheathing.

3.611.7 FIRE EXITWAYS - Fire separation walls required for the enclosure of exitways, and areas of refuge shall be constructed of masonry, reinforced concrete or any other approved non-combustible materials having the minimum fire-resistance rating prescribed by Table 115 (Part 3 Section 1) except that such walls may be constructed of combustible materials as regulated under Part 3 Section 5 3.517.23 to 3.517.26 and this section clauses 3.611.11 and 3.611.12.

3.611.8 OTHER USES/OCCUPANCIES - Fire separation walls used for subdividing purposes other than exitways and refuge shall be constructed of the types of materials and have the minimum fire-resistance rating as prescribed by Table 3.116 for the type of construction.

3.611.9 SIZE OF OPENINGS - Exitway doors located in fire separation walls shall be limited to a maximum aggregate width of twenty-five (25) percent of the length of the wall and the maximum area of any single opening shall not exceed four and one-half square metres (4.5 m^2).

3.611.10 PROTECTIVES - All opening protectives in fire separation walls shall comply with the provisions of clauses under 3.605 above and shall have the minimum fire-resistance rating as set forth in 3.617.

3.611.11 COMBUSTIBLE STAIR ENCLOSURE CONSTRUCTION - Stair enclosures constructed of approved combustible assemblies protected with component materials to afford the required fire-resistance ratings shall be continuous through combustible floor construction and shall provide an unbroken fire barrier in combination with protected floors, ceilings and fire doors, separating the exitways from the unprotected areas of the building. Such enclosures shall be firestopped to comply with 3.621.

3.611.12 WINDOWS OR BORROWED LIGHTS - In such stair enclosures, openings for the purpose of providing light shall be protected with wire glass with single panes not more than two thousand square centimetres (0.2 m^2) in area and a total area in one (1) storey of not more than five thousand square centimetres (0.5 m^2). Such light panels shall comply with the provisions under sub-section 3.619, and shall be contained in fixed sashes with frames of steel or other approved non-combustible materials.

3.611.13 CONTINUITY - All fire separation walls shall extend from the top of the fire-resistance rated floors below to the ceiling above, unless otherwise provided for this Code, and shall be securely attached thereto. Where these walls enclose required exitways, areas of refuge and shafts, or where these walls separate mixed uses, they must be continuous through all concealed spaces such as the space above a suspended ceiling, and they must be constructed tight to the underside of the floor slab or roof deck above. The supporting construction shall be protected to afford the required fire-resistance rating of the wall supported. All hollow vertical spaces shall be firestopped at every floor level as required under firestopping - 3.621.

3.612 VERTICAL SHAFTS

3.612.1 GENERAL - The provisions hereunder shall apply to all vertical shafts/enclosures, except as provided for stairway enclosures as covered in Part 3 Section 5 sub-sections 3.517 and 3.611 and under Elevator and Dumbwaiter Hoistways, Part 4 Section 3 of this Code.

3.612.2 OPEN SHAFT ENCLOSURES - The enclosing wall of shafts that are open to the outer air at the top shall be constructed of materials specified earlier in this Code for exterior walls of buildings and structures of the required fire-resistance rating specified in Table 3.114.

3.612.3 COVERED SHAFT ENCLOSURES - The enclosing walls and the top of interior covered shafts shall be constructed of approved masonry, reinforced concrete or other approved construction, with a fire-resistance rating of not less than two (2) hours, except as provided in 3.612.4 for residential buildings.

3.612.4 SHAFTS IN RESIDENTIAL BUILDINGS - In one and two-family dwellings of other than fireproof or non-combustible construction, shafts may be supported on and constructed of combustible materials or assemblies having a fire-resistance rating of not less than one (1) hour and shall extend not less than one metre (1.0m) above the roof with a ventilating skylight of non-combustible construction as specified in Exterior Trim Restrictions 3.624.

3.612.5 DUCT AND PIPE SHAFTS - In all buildings other than one- and two-family dwellings, vertical pipes arranged in groups of two (2) or more which penetrate two (2) or more floors and occupy an area of more than one thousand square centimetres ($0.1\ m^2$) and vertical ducts which penetrate two (2) or more floors, shall be enclosed by construction of not less than one (1) hour fire-resistance rating to comply with this section. All combustible pipes and ducts connecting two (2) or more stories shall be enclosed as indicated herein.

3.612.6 SHAFTS OR DUCTS NOT EXTENDING TO ROOF - Any shaft or duct that does not extend into the top storey of any building shall be enclosed with top construction of the same strength and fire-resistance rating as the floors of the building or structure in which it occurs, but not less than that of the fire-resistance rating of the shaft enclosure. Such shafts shall be provided with non-combustible vents for the relief of smoke and gases in the event of fire, with an area not less than ten (10) percent of the shaft area.

3.612.7 SHAFTS OR DUCTS EXTENDING TO OR ABOVE THE ROOF - In this case the shaft or duct shall be covered with a thermostatically controlled skylight or lid of not less than ten (10) percent of the area of the shaftway or duct, and constructed in accordance with the requirements of clauses under 3.625. The automatic operation of such lid or skylight may be controlled by fusible links designed to operate at a fixed temperature of not less than sixty degrees Celsius ($60^{\circ}C$).

3.612.8 ALTERNATE SHAFT/DUCT VENTILATION - The skylight or lid as specified in 3.612.7 may be replaced by a window, of equivalent area, in the side of the shaft above the roof, provided that the sill of such window is not less than one-half of one metre (0.5 m) above the adjoining roof; also provided that such window is equipped with an automatic vent opening, and that it is not located within six metres (6.0 m) of an opening in adjacent walls or closer than three metres (3.0 m) to the boundary or lot line.

3.612.9 BOTTOM ENCLOSURE - All shafts that do not extend to the bottom of the building shall be enclosed at the lowest level with construction of the same strength and fire-resistance rating less than that of the shaft enclosures.

3.612.10 EXISTING DUCTS OR SHAFTWAYS - In all existing ducts in buildings of assembly (occupancy group A) and institutional classifications (occupancy group 1) which are not already enclosed as herein required, the Building Authority shall direct such construction as may be deemed necessary to insure the safety of the occupants.

3.612.11 SHAFT OPENINGS - Openings other than necessary for the purpose of the shaftway shall not be constructed in shaft or duct enclosures, and all openings shall be protected with approved fire doors, fire windows or fire shutters complying with the provisions elsewhere in this Section of the Code.

3.613 Fire Resistance of Structural Members

3.613.1 REQUIREMENTS GENERALLY - The fire-resistance rating of construction assemblies and structural members shall comply with the requirements of Table 3.114 (Part 3 Section 1 of the Code) and with clauses under Fire Resistance Tests, 3.605 and onwards.

3.613.2 PROTECTION OF STRUCTURAL MEMBERS - Columns, girders, trusses, beams, lintels, or other structural members that are required to have a fire-resistance rating and that support more thn two (2) floors or one (1) floor and roof, or support a bearing wall,or a non-bearing wall more than two (2) stories high, shall be individually protected on all sides for their length of height with materials having the required fire-resistance rating. All other structural members required to have a fire-resistance rating may be protected by individual encasement, by a membrane or ceiling protection as specified in 3.614 or by a combination of both.

3.613.3 EMBEDMENTS AND ENCLOSURES - Pipes, wires, conduits, ducts or other service facilities shall <u>not</u> be embedded in the fire-protective covering of a structural member that is required to be individually encased.

3.613.4 IMPACT PROTECTION - Where the fire protection covering of a structural member is subject to impact damage from moving vehicles, the handling of merchandise, or other activity, the fire protective covering shall be protected by corner guards or by a substantial jacket of metal or other non-combustible material, to a height adequate to provide all protection, but not less than one and one-half metres (1.5 m) from the finished floor level.

3.613.5 EXTERIOR STRUCTURAL MEMBERS - Structural members located in exterior walls or along the outer lines of a building shall be protected as required by Table 3.114 for exterior bearing walls for the type of construction and shall be protected against corrosion by an approved method. The interior faces of exterior structural members shall be protected and insulated with coverings of the required fire-resistance rating specified for interior structural members in Table 3.114 (Part 3 Section 1 of this Code).

3.613.6 WALL BEAMS - Beams and girders which support walls required to have a fire-resistance rating shall be protected to afford not less than the fire-resistance rating of the wall supported, but the fire-resistance rating shall not be less than one (1) hour for members supporting masonry walls.

3.613.7 WALL LINTELS - Unless supported or suspended from construction, wall girders protected with insulating materials of the required fire-resistance rating, or when the opening is spanned by a masonry arch of the required strength, all lintels over openings in masonry walls more than two and one-half metres (2.5 m) in length shall be protected as required for structural members supporting walls for the type of construction.

3.613.8 STONE LINTELS - The use of stone lintels on spans exceeding one and one-quarter metres (1.25 m) shall not be permitted unless supplemented by fire-resistance rated structural members or masonry arches of the required strength to support the superimposed loads.

3.613.9 FIRST STOREY COLUMNS - In buildings of exterior masonry wall (Type 3) construction, required fire protection may be omitted from ground floor columns <u>supporting enclosure walls</u> located on the street boundary line.

3.614 Fire-resistance Rated Floor/roofceiling Assemblies

3.614.1 CEILING FIXTURES - Fire-resistive ceiling which constitutes an integral part of a floor or roof assembly to meet a required fire-resistance rating may have openings to accommodate non-combustible piping, ducts or electric outlets. The aggregate area of such openings in the ceiling shall be not greater than seven hundred square centimetres (0.07 m^2) in any ten square metres (10.0 m^2) of ceiling area. The fixtures and attachments shall be installed so as not to decrease the fire-resistance rating of the assembly. All duct openings shall be protected with approved non-combustible fire dampers.

3.614.2 CEILING PANELS - Where the weight of lay-in ceiling panels, used as a part of fire-resistive floor-ceiling or roof-ceiling assemblies, is not adequate to resist an upward force of five (5) kilograms per square metre (5 kg/m^2) wire ties, or other approved devices, shall be installed above the panels to prevent vertical displacement under such upward force.

3.614.3 FIRESTOPPING OF CEILING SPACES - Floor and roof construction in which the secondary structural members are not individually encased in fire-resistance rated materials or assemblies of component materials, shall be firestopped in area of not more than two hundred and seventy-five square metres (275 m^2) with non-combustible materials. Such firestopping shall comply with 3.621, or solid web structural members may be substituted for such firestops. Where floor and roof construction with accompanying ceilings is made entirely of non-combustible or fireproof construction firestopping may be omitted.

3.614.4 FIRESTOPPING OF WOOD JOIST CONSTRUCTION - Where the ceilings are suspended below wood joist floor construction, the space between the ceiling and the floor above shall be firestopped in areas of not more than one hundred square metres (100m^2) with materials meeting the requirements of sub-section 3.621.

3.614.5 LOCATION OF FIRESTOPS - Firestops shall be located directly over tenant separation walls, if the walls do not extend to the floor above.

3.614.6 UNUSABLE SPACE - In an assembly required to be of one (1) hour fire-resistance rating, the ceiling membrane may be omitted over unusable space or the flooring may be omitted where unusable space occurs above.

3.614.7 OPENINGS IN FIRE-RESISTANCE RATED FLOORS - The required fire-resistance rating of floor or floor-ceiling assemblies shall be maintained where a penetration is made for electrical, mechanical, plumbing and communication conduits, pipes and systems.

3.615 Roof Construction

3.615.1 GENERAL - All roof constructions shall be protected with non-combustible materials to afford the fire-resistance rating required by Table 3.114 (Part 3 Section 1) and/or as herein modified.

3.615.2 ROOFS OF 6 METRES OR HIGHER - When every part of the structural framework of roofs in Type 1 or Type 2 buildings is six metres (6 m) or more above the floor immediately below, all fire protection of the structural members may be omitted, including the protection of trusses, roof framing and decking. Heavy timber members, in accordance with Part 3 Section 1 (3.117) may be used for such unprotected members in one (1) storey buildings.

3.615.3 EXCEPTION - Buildings of H (High Hazard), S-1 (Moderate Hazard Storage) or M (Mercantile) occupancies when of Type 1 or 2-A construction shall not have less than one (1) hour fire-resistance rated roof construction.

3.615.4 ROOF SLABS, ARCHES AND DECKING - Where the omission of fire protection from roof trusses, roof framing and decking is permitted the horizontal or sloping roofs in Type 1 and Type 2 buildings, immediately above such members, shall be constructed of non-combustible materials of the required strength without a specified fire-resistance rating, or of mill type construction in buildings not over five (5) stories or twenty metres (20 m) in height.

3.615.5 FIRESTOPPING - Firestopping of ceiling and attic spaces shall be provided as required under this Code (see 3.614 and 3.621.)

3.616 Exterior Opening Protectives

3.616.1 WHERE REQUIRED - Where specified herein, the exterior openings of all buildings and structures other than churches (Occupancy group A-4), residential buildings (occupancy groups R-2 and R-3), buildings of unprotected non-combustible (Type 2-C) construction, and buildings of frame (Type 4) construction shall have approved opening protectives meeting the requirements of this Code and the provisions of Part 3 Section 3 for special uses and occupancies.

3.616.2 HORIZONTAL EXPOSURE - Approved protectives shall be provided in every opening where the fire separation is less than five metres (5 m).

3.616.3 VERTICAL EXPOSURE - Approved protectives shall be provided in every opening which is less than fifteen metres (15 m) vertically above the roof of an adjoining or adjacent structure that is within a horizontal distance of nine metres (9 m) of the wall in which the opening is located, unless such roof construction affords a fire-resistance rating of not less than one and one-half (1 1/2) hours.

3.616.4 GROUND FLOOR OPENINGS - The required fire-resistance rated opening protectives may be omitted in ground floor openings facing on a street or other public space, lane, or right-of-way not less than nine metres (9 m) wide, and when extending no more than eight metres (8 m) above ground level.

3.616.5 PROTECTED OPENINGS - Required protective assemblies in exterior openings may be fixed, they may be self-closing, or they may be provided with automatic self-closing devices approved by the local Fire Authority.

3.616.6 UNPROTECTED OPENINGS - Where a fire-resistance rating is not required by this section for openings in exterior walls, windows and doors may be of unprotected wood. Glazing shall conform to the requirements of this Code.

3.617 Fire Doors

3.617.1 FIRE DOOR ASSEMBLIES - Approved fire door assemblies as defined in this Code shall be constructed of any material or an assembly of component materials which meets the test requirements of Part 3 Section 6, 3.605 and 3.606 of this Code and the fire-resistance ratings herein required, unless otherwise specifically provided for in this Code.

3.617.2 LABELLED OR CERTIFIED PROTECTIVE ASSEMBLIES - Protective assemblies meeting the requirements of 3.605.10 and 3.605.11 and relevant ASTM, or BSI Standards, including shop inspection shall be approved for use as provided for in this Code.

3.617.3 MULTIPLE DOORS IN FIRE WALLS - Two (2) doors, each with a fire-resistance rating of one and one-half (1 1/2) hours, installed on opposite sides of the same opening, shall be deemed equivalent in fire-resistance rating to one (1) three (3) hour fire door.

TABLE 3.617

FIRE DOOR RESISTANCE RATINGS, IN HOURS

Location of Fire Door	Fire-resistance Rating in Hours
Fire Walls and fire separation walls of three (3) or more hour construction	3
Fire walls, fire separation walls and exitway enclosures of two (2) hour construction	1 1/2
Shaft enclosures and elevator hoistways of two (2) hour construction	1 1/2
Shaft enclosures of one (1) hour construction	1
Fire separation walls of one (1) hour construction	3/4

3.617.4 MULTIPLE DOORS IN FIRE SEPARATION WALLS - Two (2) doors, of three-quarter (3/4) hour fire-resistance rating each, installed on opposite sides of the same opening shall be deemed equivalent in fire-resistance rating to a one and one-half (1 1/2) hour fire door; except when used in a required exitway.

3.617.5 GLASS PANELS - Wired glass panels shall be permitted in fire doors within the limitations of sub-section 3.619 below and as herein specifically prescribed.

3.617.6 CLOSING DEVICES - Except as may be otherwise provided for openings in fire walls and fire separation walls, all fire doors shall be self-closing and shall be closed during occupancy of the building or part thereof. The building official may accept the use of rate of rise heat actuated devices meeting the requirements of the approved rules on doors that are normally required to be open for ventilation or other specified purposes when the safety of the occupants is not endangered thereby.

3.618 Fire Windows and Shutters

3.618.1 FIRE-RESISTANCE RATING - Approved assemblies of fire windows and fire shutters shall meet the requirements of 3.605 and 3.606 or they shall be approved certified (or labelled) assemblies meeting all the requirements of 3.605.11.

3.618.2 EXCEPTION - Steel, aluminium, or bronze window frame assemblies of a gauge giving a solid metal thickness of not less than three and one-fifth millimetres (3.2 mm) for both sashes and frames, and made to take glazing of not less than six millimetres (6 mm) wired glass when securely installed in the building construction, shall be deemed to meet the requirements for a three-quarter (3/4) hour fire window assembly.

3.618.3 (FIRE) WINDOW MULLIONS - All metal window mullions (and frames) which exceed a nominal height of three and one-half metres (3.5m) and are of hollow section shall be protected with insulating materials to afford the same fire-resistance rating as required for the wall construction in which the window is located.

3.618.4 SWINGING (HINGED) FIRE SHUTTERS - When fire shutters of the hinged or swinging type are used in exterior openings, not less than one (1) row in every three (3) vertical rows shall be arranged to be readily opened from the outside and shall be identified by distinguishing marks or letters not less than 150 mm high.

3.618.5 ROLLING FIRE SHUTTERS - When fire shutters of the rolling type are used, they shall be of approved counterbalance construction that can be readily opened from the outside.

3.619 Wired Glass

3.619.1 GENERAL - SIZES OF WIRED GLASS GLAZING ALLOWABLE -As a guide to the use of wired glass in fire doors, in windows, in borrowed lights, and even in skylights, standards in many countries have based their regulations on the "old" quarter-inch rough rolled (plate) or clear plate single wired glass. This glass now becomes "6 mm wired glass" and the following table is based on this weight or thickness wherever the glass is made or from wherever it is imported.

3.619.2 There is no restriction in this Code for designers who wish to use a heavier weight (greater thickness) for single or double wired glass where other factors, such as vibration sound insulation by using double glazing, or where special cases arise for added protection of workmen or even for larger panes than allowed in the following Table, provided reasons are submitted for the approvals given by the Local and/or Fire Authority concerned.

TABLE 3.619

LIMITING SIZE FOR 6mm SINGLE WIRED (GLAZING) PANELS

Rating Opening	Max. Area	Max. Height	Max. Width
3 hr. Class A door	0	0	0
1 and 1 1/2 hr. Class B doors	645 cm^2	210 cm^2	65 cm^2
3/4 hr. Class C door	8360 cm^2 (0.836 m^2)	350 cm^2	350 cm^2
1 1/2 hr. Class D door	0	0	0
3/4 hr. Class E door	8360 cm^2 (0.836 m^2)	350 cm^2	350 cm^2
Fire Windows	8360.cm^2 (0.836 m^2)	350 cm^2	350 cm^2

3.619.3 FIRE WALLS - Wired glass in fire doors located in fire walls is prohibited, except where serving as horizontal exits, the self-closing swinging doors may be provided with a narrow, preferably vertical, vision panel of not more than six hundred and forty-five square centimetres (645 cm^2) without either dimension exceeding three hundred millimetres (300 mm).

3.619.4 FIRE SEPARATION WALLS - Wired glass vision panels may be used in fire doors of one and one-half (1 1/2) hour fire-resistance rating intended for use in fire separation walls; but the glass panels shall not be more than six hundred and forty-five square centimetres (645 cm^2) as for fire walls above.

3.619.5 EXITWAY PROTECTIVES - Unless specifically required in Part 3 Section 3 of this Code to be solid in such locations where unusually hazardous conditions prevail, fire doors in elevator and stairway shaft enclosures may be equipped with approved wire glass vision panels which shall be so located as to furnish clear vision of the passageway or approach to the elevator or stairway. Such vision panels shall not exceed the size limitations specified in Class B doors.

3.619.6 FIRE SEPARATION WALLS - Six millimetre (6 mm) wired glass panels may be used in fire separation walls used for subdividing purposes as set forth in 3.611.7, provided the required fire-resistance rating of the wall does not exceed one (1) hour. The maximum size of such panels shall not exceed the limiations for a three-quarter (3/4) hour Class C door.

3.620 **Fire-Resistive Requirements for Plaster (Rendering)**

3.620.1 THICKNESS OF PLASTER - The required thickness of fire-resistance rated plaster protection shall be determined by the prescribed fire tests for specified use and type of construction and in accordance with the provisions of interior plastering and for exterior plastering specified in this Code. The thickness in all cases shall be measured from the face of the lath when applied to fibre board, wood, or gypsum lath and from the back of metal lath.

3.620.2 PLASTER EQUIVALENTS - For fire-resistive purposes, twelve millimetres (12 mm) of unsanded gypsum plaster shall be deemed equivalent to 18 millimetres (18 mm) of one (1) to three (3) sanded gypsum, or 25 millimetres (25 mm) portland cement sand plaster.

3.620.3 NON-COMBUSTIBLE BACKING - In fireproof (Type 1) and non-combutible (Type 2) construction, plaster shall be applied directly on to concrete, masonry or onto approved non-combustible plastering base and furring.

3.620.4 DOUBLE REINFORCEMENT - Except in solid plaster partitions, or when otherwise determined by the prescibed fire tests, plaster protections more than 25 millimetres in thickness shall be reinforced with an additional layer of approved metal lath imbedded at least 18 millimetres from the outer surface and fixed securely in place.

3.620.5 PLASTER ALTERNATES FOR CONCRETE - In reinforced concrete construction, gypsum or portland cement plaster may be substituted for 12 mm of the required poured concrete construction except that a minimum thickness of 100 millimetres of poured concrete shall be provided in all reinforced concrete floors and 25 millimetres in reinforced concrete columns, in addition to the plaster finish and the concrete base shall be prepared in accordance with that section of this Code dealing with Walls and Wall Thickness.

3.621 **Fires opping**

3.621.1 WHERE REQUIRED - Firestopping shall be designed and constructed close for all concealed draft openings and to form effectual fire barriers against the spread of flame fire between stories of every building and in all open structural spaces therein, including the following locations: for the subdivision of attic spaces, for combustible wall, partition and floor framing, for ceiling space (see 3.614), for open spaces behind accoustical and other finishes, for floor sleeper space, and for pipe, duct and flue openings in the mechanical section (see 3.623 in Part 4 of this Code.

3.621.2 FIRESTOPPING MATERIALS - All firestopping shall consist of approved non-combustible materials securely fastened in place, firestops of two (2) thicknesses of 25 mm lumber with broken lap joint or one thickness 9.5 mm or 10 mm plywood with joints backed by 9.5 mm or 10 mm plywood or of 50 mm lumber installed with tight joints shall be permitted in open spaces of wood framing.

3.621.3 REQUIRED INSPECTION - Firestopping shall not be concealed or covered from view until inspected and approved by the Building Inspector or other Local Authority representative.

3.622 Interior Finish and Trim (and Application of Same)

3.622.1 GENERAL - Interior finish and interior trim of buildings shall conform to the requirements of this Section. Interior finish shall include all dadoes and panelling or other finish applied structurally or for accoustical treatment, insulation, decoration, or similar purposes. The use of a surface finish of paper, or of material of not greater fire hazard than paper, shall not be prohibited provided such finish does not exceed one millimetre (1.0 mm) in thickness, and is applied directly to a non-combustible base or substrate meeting the requirements of 3.605.15. Show windows in the ground floor of buildings may be of wood or unprotected metal framing.

3.622.2 EXPOSED CONSTRUCTION - These requirements shall not be considered as requiring the installation of interior finish, but where construction or fire protection materials are exposed in rooms or spaces used for the occupancies specified, the hazard from rate of flame-spread of such exposed materials shall be not greater than that of the interior finish permitted for such occupancy or use. Exposed portions of structural members complying with the requirements for heavy timber type construction in sub-section 3.117 onwards in Part 3 Section 1 of the Code shall not be subject to interior finish regulations.

3.622.3 SMOKE OR GLASS - Interior finish materials shall not be permitted that have a smoke-developed factor greater than four hundred and fifty (450) when tested in accordance with the method of test for surface burning characteristics of building materials. When restrictions are not otherwise established in this Code, interior finish is not controlled, except that pyroxylin or similar finishes shall not be applied which, as dry films, produce excessive smoke or toxic fumes when exposed to fire.

3.622.4 MATERIALS - Material may be used for interior finish and trim only as specifically provided in this Code for the occupancy or use of the space in which it is installed.

Use of any material for floor finish, interior finish, and trim in a building of Type 1 or Type 2 construction within the scope permitted in this section or under 3.623 shall not declassify the building with respect to its type of construction.

3.622.5 FOAM PLASTICS - Foam plastics shall not be used as interior finish or as interior trim except as provided under 3.622.9.

3.622.6 INTERIOR FINISH - Interior finish of wall and ceilings shall have a flame-spread rating not greater than that designated by the class prescribed for the various occupancy groups listed in Table 3.622 when tested in accordance with the requirements of 3.606.

3.622.7 BASEMENTS - In buildings, other than one and two-family residences, Class 1 or Class II interior finish shall be used in all basements or other underground spaces from which there is no direct access to outside of the building; provided that the occupancy is for any purpose other than storage or service facilities.

3.622.8 MAXIMUM FLAME SPREAD - Interior finish materials with flame-spread classifications in excess of two hundred (200) shall not be used in any room or space subject to human occupancy, except to such extent as may be specifically permitted by the Local Authority on the basis of a finding that such use does not significantly increase the life hazard.

3.622.9 INTERIOR TRIM - Baseboards, chair-rails, mouldings, trim around openings and other interior trim, not in excess of ten (10) percent of the aggregate wall and ceiling areas of any room or space, may be of Class I, II or III materials, except that in trim around fire windows and fire doors shall comply with the requirements of 3.617 and 3.618.

3.622.10 APPLICATION/ATTACHMENT OF INTERIOR FINISHES - When interior finishes are regulated by the requirements of this Code, interior finish materials shall be applied or otherwise fastened in such a manner that they will not readily become detached when subjected to room temperatures of 95^{o}C or more for thirty (30) minutes, or otherwise become loose through changes in the setting medium from the effects of time or conditions of occupancy.

3.622.11 APPLICATION TO STRUCTURAL ELEMENTS - Interior finish materials applied to walls, ceilings, or structural elements of a building which are required to be fire-resistance rated or to be constructed of non-combustible component materials, shall be applied directly against the exposed surface of such structural elements, or to furring strips attached to such surfaces with all concealed spaces created thereby firestopped where in excess of one square metre (1.0 m^2) or two and one-half metres (2.5 m).

3.622.12 FURRED CONSTRUCTION - Where walls, ceilings or other structural elements are required to be fire-resistance rated or to be constructed of non-combustible component materials and interior finish is set out or dropped distances greater than one and three-quarter (1 3/4) inches from the surface of such elements, only material of which both faces qualify as Class I shall be used, unless the finish material is protected on both sides by an automatic fire suppression system (see Note 1 to Table 3.622) or is attached to a non-combustible backing, complying with 3.622.15 below, or to furring strips applied directly to such backing as provided in 3.622.11.

3.622.13 HEAVY TIMBER CONSTRUCTION - Interior finish materials may be applied directly to the wood members and decking of heavy timber (Type 3-A) construction, where permitted, or to furring strips applied to such members or wood decking as provided in 3.622.11.

TABLE 3.622

INTERIOR FINISH REQUIREMENTS

Occupancy Groups		Required Vertical Exitways and Passageways (4)	Corridors Providing Exitways Access	Rooms Enclosed Spaces (1)	
A-1	Assembly, theatres	1	1	11	(2)
A-2	Assembly, night clubs	1	1	11	(2)
A-3	Assembly, halls, terminals, restaurants	1	1	11	(2)
A-4	Assembly, churches, schools	1	1	111	
B	Business	1	11	111	
F	Factory and industrial	1	11	111	
H	High Hazard	1	11	111	
I-1	Institutional restricted	1	1	1	(3)
I-2	Institutional incapacitated	1	11	1	(3)
M	Mercantile walls, ceilings	1	11	11	(5)
R-1	Residential, hotels	1	11	111	
R-2	Residential, multi-family dwellings	1	11	111	
R-3	Residential, one and two-family dwellings	111	111	111	
S-1	Storage, moderate hazard	1	11	111	
S-2	Storage, low hazard	1	11	111	

NOTE 1 - Requirements for rooms or enclosed spaces are based upon spaces enclosed in partitions of the building or structure, and where fire-resistance rating is required for the structural elements the enclosing partitions shall extend from the floor to the ceiling. Partitions which do not comply with this shall be considered as enclosing spaces and the rooms or spaces on both sides thereof shall be counted as one. In determining the applicable requirements for rooms or enclosed spaces, the specific use or occupancy thereof shall be the governing factor, regardless of the occupancy group classification of the building. When an approved automatic fire suppression system is provided the interior finish of Class II or III materials may be used in place of Class I or II materials respectively, where required in the Table.

NOTE 2 - Class III interior finish materials may be used in place of assembly with a capacity of three hundred (300) persons or less.

NOTE 3 - Class III interior finish materials may be used in administrative areas, Class II interior finish materials may be used in individual rooms of not over four (4) persons capacity. Provisions in Note 1 allowing a change in interior finish classes when fire suppression protection is provided shall not apply.

NOTE 4 - Class III interior finish materials may be used for dadoes or panelling for not more than ninety square metres (90 m^2) of applied surface area in the grade lobby when applied directly to a non-combustible base or over furring strips applied to a non-combustible base and firestopped as required under 3.622.10 to 3.622.13.

NOTE 5 - Class III interior finish materials may be used in mercantile occupancies of two hundred and fifty square metres (250 m^2) or less gross area. Used for sales purposes on the ground floor only. (Balcony permitted).

3.622.14 CLASS II AND III MATERIAL - Interior finish materials, other than Class 1 material, which are less than six millimetres (6 mm) in thickness shall be applied directly against a non-combustible backing or a backing complying with the requirements of 3.605.15 unless the tests under which such material has been classed were made with the materials suspended from the non-combustible backing.

3.622.15 BACKING MATERIAL (s) - Backing for interior finish materials shall be continuous surface with permanently tight joints, equal in area to the area of the finish, and extending completely behind such finish in all directions; and may be of any materials meeting the requirements of this code for non-combustible classification of material under clause 3.605.13 or of fire-retardant-treated wood. When the backing does not constitute an integral part of the structural elements or system it shall be attached directly to the structural elements or to furring strips as required for the application of finish according to 3.622.11 or may be suspended from the structural members at any distance provided concealed spaces created thereby shall be firestopped in accordance with the applicable requirements of this Code. Where Class III interior finish is applied to a continuous non-combustible backing beneath wood joist construction, the allowable area for firestopping required in 3.614.4 may be increased to two hundred and fifty square metres (250 m^2).

3.623 Combustible Materials Permitted in Floor Construction of Types 1 and 2 Buildings

3.623.1 GENERAL - Except as provided in Means of Egress, Part 3 Section 5 (Interior Exitways Stairways 3.517) and in Special Use and Occupancy Requirements, Part 3 Section 3 (Places of Public Assembly) for threatres and similar places of public assembly (Occupancy Groups A-1 and A-2) the use of combustible materials in or on floors of Type 1 and Type 2 buildings shall be as herein specified.

3.623.2 SLEEPERS, CLEATS, NAILING BLOCKS OR GROUNDS - Floor sleepers, cleats, nailing, blocks and grounds may be constructed of combustible materials, provided the space between the fire-resistance rated floor construction and the flooring is either solidly filled with non-combustible materials or firestopped in areas of not more than ninety-five square metres (95 m^2), provided such open spaces shall not extend over or through permanent partitions or walls.

3.623.3 FLOORING - Wood finish floorings may be attached directly to the embedded or firestopped wood sleepers and wood finish flooring shall be permitted when cemented directly to the top surface of approved fire-resistance rated construction or cemented directly to a wood subfloor attached to sleepers as provided in 3.623.2. Combustible insulating boards not more than 12 millimetres thick and covered with approved finished flooring may be used for deadening or heat insulating when attached directly to a non-combustible floor assembly or to wood subflooring attached to sleepers as provided in clause 3.623.2 above.

3.624 Decorative Materials and Exterior Trim Restrictions

3.624.1 GENERAL - In places of public assembly, all draperies, hangings, and other decorative materials suspended from walls or ceilings shall be non-combustible or flame-resistant meeting the requirements of 3.606 as herein specified.

3.624.2 NON-COMBUSTIBLE - The permissible amount of non-combustible decorative hangings shall not be limited.

3.624.3 FLAME-RESISTANT - The permissible amount of flame-resistant decorative hangings shall not exceed ten (10) percent of the total wall and ceiling area.

3.624.4 EXTERIOR TRIM: GUTTERS AND VALLEYS - All gutters and valleys hereafter placed on, or in roof construction of buildings other than frame (Type 4) buildings, one and two-family dwellings, and private garages, and any similar accessory buildings shall be constructed of non-combustible materials.

3.624.5 ARCHITECTURAL TRIM - All architectural trim, such as cornices and other exterior architectural elements attached to the exterior walls of buildings of Types 1 and 2 construction shall be constructed of approved non-combustible materials and shall be secured to the wall with metal or other approved non-combustible brackets; except that outside the fire limits, such trim may be of non-combustible material when the building does not exceed three (3) stories or twelve metres (12.0 m) in height. Combustible trim may be used on all buildings of Types 3 and 4 construction.

3.624.6 LOCATION - When combustible architectural trim is located along the top of exterior walls it must be completely backed up by the exterior wall and shall not extend over or above the top of exterior walls.

3.624.7 FIRESTOPPING - Continuous exterior architectural trim constructed of combustible materials shall be firestopped as required in subsection 3.621.

3.624.8 COMBUSTIBLE HALF-TIMBERING - In buildings of masonry Type 3 construction that do not exceed three (3) stories or twelve metres (12.0 m) in height, exterior half-timbering and similar architectural decorations may be constructed of wood or other equivalent combustible materials, provided such trim is backed up solidly with approved non-combustible materials.

3.624.9 BALCONIES - All balconies attached to or supported by buildings of Types 1 and 2 construction shall be constructed of non-combustible materials. Balconies attached to or supported by buildings of Types 3 and 4 construction may be of unprotected non-combustible materials or frame construction. Balconies of frame construction shall afford the fire-resistance rating required by Table 3.114 for floor construction and the aggregate length shall not exceed fifty (50) percent of the building perimeter on each floor.

3.624.10 BAY AND ORIEL WINDOWS - All bay and oriel windows attached to or supported by walls other than frame construction shall be of non-combustible materials, unless specifically exempted by Part 3 Section 2 of this Code.

3.624.11 EXISTING COMBUSTIBLE CONSTRUCTION - Any existng cornice or other exterior architectural element constructed of wood or similar combustible materials may be repaired with the same material to the extent of fifty (50) percent of its area in any one (1) year if the public safety is not thereby endangered.

3.624.12 WOOD VENEERS - Inside the fire limits wood veneers are permitted in accordance with Part 3 Section 2.

3.625 Roof Structures

3.625.1 GENERAL - All construction, other than aerial supports, clothes dryers and similar structures less than four metres (4.0 m) high, water tanks and cooling towers as hereinafter provided and flag poles, erected above the roof of any part of any building or structure located within the fire limits or of any building or structure more than twelve metres (12.0 m) in height outside the fire limits shall be constructed of non-combustible materials.

3.625.2 ROOF ACCESS TRAPS OR TRAPDOORS - Trapdoors and other access ways on to any roof area as required by Part 3 Section 5 (Means of Egress) sub-section 3.518 shall be not less than one metre (1.0 m) by six-tenths of one metre (0.6 m) in size and shall be of fire-resistance-rated construction in fireproof (Types 1-A and 1-B) and non-combustible (Type 2) buildings, and of approved non-combustible materials. They may however be of wood covered on top and edges with sheet metal in exterior masonry (Type 3) and protected frame (Type 4-A) buildngs.

3.625.3 SKYLIGHTS - SASHES AND FRAMES - Sashes and frames of all skylights on buildings of types 1 and 2 construction shall be of steel, metal, or other approved non-combustible materials. In foundries or buildings where acid fumes deleterious to metal are incidental to the use of the building, treated wood or other specialist approved non-corrosive materials shall be permitted.

3.625.4 GLASS, WIRED OR PLAIN - Skylights shall be glazed with wired glass not less than 6 mm in thickness or shall be approved glass block construction conforming to this Code, except the skylights placed over shafts and stair enclosures and skylights used for emergency heat and smoke ventings shall be glazed with plain glass not over three millimetres (3.0 mm) in thickness. A single panel of wired glass in skylights shall not exceed 4650 cm^2 (0.465 m^2) in area or 1.2 m in any dimension. Light-transmitting plastic may be used if it complies with non-combustibility tests required by this Code or relevant British or other approved Standards.

3.625.5 SCREENS - Plain glass skylights shall be protected by substantial corrosion-resistive metal or other approved non-combustible screens having a mesh not less than 19 mm by 19 mm nor larger than 25 mm by 25 mm, constructed of not lighter than 2.05 mm wires. The screen shall be erected at a distance of not less than 100 millimetres nor more than 250 millimetres above all glazed portions of skylight and shall project on all sides for a distance of not less than the height of the screen above the glass. A similar screen shall be placed below the skylight to afford protection to the occupants of the building. The provisions for wired glass or screen protection shall not apply to glass block skylights or to greenhouse construction.

3.625.6 PENTHOUSES - Penthouses shall be considered as part of the next lower storey, and the enclosure shall conform to the requirements for exterior walls of the building type as regulated by Table 3.114 (Part 3 Section 1 of this Code) except as modified hereunder.

3.625.7 RECESSED WALLS - When the exterior wall of a penthouse is recessed or stepped back from the exterior wall of the next lower storey by one and one-half metres (1.5 m) or more and the exterior wall of the next lower storey is required to have a fire-resistance rating of greater than one and one-half (1 1/2) hours, the penthouse exterior wall may be constructed with a fire-resistance rating of not less than one and one-half (1 1/2) hours, covered on the outside with non-combustible, weatherproof material and supported on protected steel or reinforced concrete construction.

3.625.8 DOORS, FRAMES AND SASHES - Doors, frames and window sashes, except where otherwise specifically required to be fireproof or fire-resistance rated under this Code, shall be constructed the same as other similar elements in the building.

3.625.9 OTHER ENCLOSED ROOF STRUCTURES - Enclosed roof structures, other than the penthouses as defined herein shall be considered a storey of the building and the enclosure shall conform to the requirements for exterior walls of the building type as regulated by Table 3.114 in Part 3 Section 1, and the provisions described in the following clauses 3.625.10 and 3.625.11.

3.625.10 NON-COMBUSTIBLE MATERIALS - Unless constructed of masonry or reinforced concrete, roof structures erected on buildings of fireproof or non-combustible (Types 1 or 2) constructions shall be enclosed in walls of non-combustible materials having a fire-resistance rating of not less than one (1) hour, protected with weather-resistive roof coverings complying with sub-section 3.626 onwards.

3.625.11 COMBUSTIBLE MATERIALS - Roof structures erected on the roof of exterior masonry buildings (Type 3) and protected frame buildings (Type 4-A) may be constructed of combustible materials protected to afford a one (1) hour fire-resistance rating covered on the outside with approved roofing materials.

3.625.12 MANSARD OR OTHER HIGH PITCHED ROOFS - Every mansard or other steeply sloping (high pitched) roof having a pitch of more than sixty (60) degrees to the horizontal hereafter erected on any building or structure of other than Type 4 frame construction more than three (3) stories or twelve metres (12.0 m) in height shall be constructed of non-combustible materials with a fire-resistance rating of not less than one (1) hour; except that when the building is more than seven (7) stories or twenty-five metres (25.0 m) in height, such roofs shall afford the same fire-resistance rating required for the exterior walls of the building but need not exceed one and one-half (1 1/2) hour fire-resistance rating.

3.625.13 LOW SLOPE (PITCHED) ROOFS - When the pitch is less than sixty (60) degrees to the horizontal, the mansard roof or other sloping roof located on any building may be constructed of the same materials as required for the roof of the building.

3.625.14 DORMERS - The sides and roofs of dormers shall be of the same type of construction as the main roof construction; except that where a side of the dormer is merely a vertical extension of an exterior wall it shall be subject to the same fire-resistance rating requirements as apply to the wall of the building. The roofs of the dormers shall be protected with approved roof coverings complying with subsection 3.626 onwards. The sides of dormers shall be protected with approved roof coverings or with materials which would be permitted for covering the exterior walls of the buildings.

3.625.15 WATER TANKS AND SUPPORT FOR SAME - Water tanks having a capacity of more than two thousand, two hundred and fifty litres (2250.0 l) placed in or on a building shall be supported on masonry, reinforced concrete, steel or other approved non-combustible framing or on timber conforming to heavy timber mill construction (Type 3-A); provided that, when such supports are located in the building above the lowest floor, they shall be fire-resistance rated as required for fireproof (Type 1-A) construction.

3.625.16 EMERGENCY DISCHARGE FROM SUCH TANKS - A pipe or outlet shall be located in the bottom or in the side close to the bottom, or the tank shall be fitted with a quick-opening valve to enable the contents to be discharged in an emergency to a suitable drain complying with the Plumbing Code in Part 4 Section 4 of this Code.

3.625.17 LOCATION - A tank shall not be located over or near a stairway or elevator shaft unless a solid roof or floor deck is constructed underneath the tank.

3.625.18 TANK COVERS - All enclosed roof tanks exposed to the weather shall have approved covers sloping towards the outer edges (see also under Plumbing and Drainage, Part 4 Section 4 of this Code).

3.625.19 METAL STRAP PROTECTION - Where hoop iron or other type of ferrous metal straps are used in the construction of tank stands, or for holding down tanks on to stands on any roof (or other exposed position) they shall be protected with approved corrosion-resistant coatings.

3.625.20 COOLING TOWERS (a) LOCATED WITHIN FIRE LIMITS - Within the fire limits, cooling towers erected on the roofs of buildings shall be constructed of non-combustible materials, except that drip bars may be of wood. Cooling towers may be constructed entirely of fire-retardant-treated wood, including drip bars.

3.625.21 COOLING TOWERS (b) LOCATED OUTSIDE FIRE LIMITS - Outside the fire limits, cooling towers may be constructed of wood or other approved materials of similar combustible characteristics; except that when the base of the tower is more than sixteen and one-half metres (16.5 m) above ground level and the tower is located on a building, the drip bars only may be fabricated of combustible materials as herein provided.

3.625.22 MISCELLANEOUS ROOF STRUCTURES - Except as herein specifically provided, all towers, spires, dormers, or cupolas shall be erected of the type of construction and fire-resistance rating required for the building to which they are accessory as regulated by Table 3.114 (Part 3 Section 1) and Part 3 Section 2; except that when the height of such appurtenant structures exceeds twenty-five metres (25.0 m) above ground level or when the area at any horizontal section of the tower spire dormer or cupola exceeds twenty square metres (20 m^2) or when it is used for any purpose other than as a belfry or architectural embellishment, the structure and its supports shall be of fireproof (Type 1) construction, non-combustible (Type 2) construction, or fire-retardant-treated wood complying with 3.605.14 and 3.605.15 of this section of the Code. Radio and television towers and antennae shall be constructed to comply with Part 3 Section 3 of this Code (Radio and Television Towers and Antennae).

3.626 Roof Coverings

3.626.1 CLASSIFICATION - All approved roof coverings shall meet the test requirements and be classified in accordance with clauses 3.605.5 to 3.605.8 of this Code.

3.626.2 EXISTING ROOFS - The repair of existing roofs shall comply with the provisions of Part 1 Section 1 (Administration and Enforcement) of this Code but more than twenty-five (25%) percent of the roof covering of any building shall not be replaced in a period of twelve (12) months unless the entire roof covering is made to conform to the requirements for new roofing.

3.626.3 CLASSIFICATION OF USE - (See under 3.605.4 also for classifications A, B, C and unclassified.

3.626.4 CLASS A ROOF COVERINGS - Class A roof coverings shall be permitted for use in buildings of all types of construction.

3.626.5 CLASS B ROOF COVERINGS - Class B roof coverings shall be permitted as the minimum for use in buildings and structures or Type 1 construction.

3.626.6 CLASS C ROOF COVERINGS - Class C roof coverings shall be permitted as the minimum for use in buildings and structures of Types 2, 3 and 4A construction.

3.626.7 NON-CLASSIFIED ROOF COVERINGS - Non-classified roof coverings shall be permitted on the buildings or structures as listed below.

(a) Buildings and structures of unprotected frame (Type 4-B) construction when the distance from any other building is not less than three and one-half metres (3.5 m).

(b) Private garages, airplane hangers and similar accessory structures, not exceeding one (1) storey or six metres (6 m) in height, and five hundred (500 m^2)

square metres in area, when outside the fire limits, located in the same lot with a dwelling and with a fire separation of not less than three and and one-half metres (3.5 m).

(c) Moderate and low hazard storage buildings (occupancy groups S-1 and S-2) not exceeding one (1) storey or six metres, (6.0 m) in height and six hundred square metres (600 m^2) in area with a fire separation of not less than three and one-half metres (3.5 m). Fire walls may be used to obtain the required fire separation.

3.626.8 ROOF INSULATION - The use of cork, fibre or other insulating boards, and/or other combustible roof insulation is permitted provided that it is covered with approved roof coverings applied directly to such insulation.

3.626.9 EARTHING/GROUNDING OF METAL ROOFS - Whenever, because of hazard resulting from electrical equipment, or apparatus located thereon, or because of lighting, or proximity to power lines, or for any other reason, it is deemed necessary by the Building Authority, metal roofs shall be grounded by bonding together each course or strip and the bonding conductor or conductors shall be extended to and attached in an approved manner to the grounding electrode used to ground the electrical system within the building on which such metal roofing is applied. The conductors used to bond courses or strips of metal roofing together, or any conductor extended for grounding earthing to the grounding electrode, shall not have greater resistance than the conductor used to ground the electrical system within the building.

3.626.10 ALTERNATE METHODS OF EARTHING/GROUNDING OF METAL ROOFING - Alternative methods of grounding roofing may be used, provided they are at least equal in performance to the methods described herein, and further provided that such desired method is first submitted to and approved by the Building Authority and/or its Electrical Inspectors.

3.626.11 ASPHALT SHINGLE APPLICATION - Asphalt shingles laid with double coverage may be installed on slopes below 1 in 3 (10 cm rise to 30 cm rise) to as low as 1 in 6 pitch, provided the shingles are approved self-sealing shingles or are hand-sealed and are installed with an underlay consisting of two layers of No. 15 bituminised felt, applied shingle fashion. (Horizontally along the roof).

3.626.12 RE-ROOFING IN ASPHALT SHINGLES - Not more than two (2) overlays of asphalt shingles shall be applied over an existing asphalt shingle roof. Not more than two (2) overlays of asphalt shingles applied over wood shingles shall have an underlay of not less than Type 30 non-perforated bituminous felt.

3.626.13 WOOD SHAKE APPLICATION - Not more than one (1) overlay of wood shakes shall be applied over an existing asphalt shingle or wood shingle roof. One (1) layer (or strip) of 45 cm wide Type 30 felt shall be interlaced between each layer of shakes.

3.626.14 APPLICATION OVER SHAKES - New roof covering shall <u>not</u> be applied over an existing shake roof.

3.626.15 FLASHING AND EDGINGS - Rusted or damaged flashings, vent caps, and metal edgings shall be replaced with new materials as necessary when any type of re-roofing is being carried out.

CARIBBEAN UNIFORM BUILDING CODE

PART 3
OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 7

FIRE PROTECTION SYSTEMS

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SECTION 7

3.700 FIRE PROTECTION SYSTEMS

3.701 **Scope**

3.701.1 The provision of this Section shall specify where fire protection systems are recommended in all buildings or structures or parts thereof. The occupancy classification of the building to be protected (see Part 3, Section 1 of this Code Occupancy and Construction Classifications and Public Health) has an important bearing on the type of protection system required for that building.

3.701.2 INSTALLATION REQUIREMENTS - The installation methods, repair operation and/or maintenance of fire protection systems shall be in accordance with this Code.

3.701.3 MAINTENANCE - The owner, tenant or lessee of every building or structure shall be responsible for the care and maintenance of all fire protection systems, including equipment and devices to insure the safety and welfare of the occupants. Fire protection systems shall not be disconnected or otherwise rendered unserviceable without first notifying the Fire Department.

3.701.4 When installations of required fire protection systems are interrupted for repairs, or other reasons, the owner, tenant or lessee shall immediately inform the Fire Department and shall diligently prosecute the restoration of the protection.

3.701.5 MATERIAL AND EQUIPMENT - All materials and equipment used in a fire protection system shall be approved, consistent with the requirements of this Code, and other relevant Standards such as the U.S. National Fire Protection Association (NFPA) 73-76.

3.701.6 THREADS - All couplings provided for Fire Department connections to sprinkler systems, standpipes, yard hydrants, or any other fire hose connections shall be uniform with those used by the local Fire Department.

3.701.7 SIGNS - If fire suppression control valves are located in a separate room or building, a sign shall be provided on the entrance door. The lettering for such a sign shall be of a conspicuous colour, and shall be not less than 100 mm in height, and shall read "Sprinkler Control Valves" and/or "Standpipe Control Valves" or indicate any other types of system. (See also Part 3, Section 8 for additional signs).

3.701.8 TESTS - Where required by this article, and any standards listed herein, all flow test connections and points of fluid discharge shall be reasonably accessible and acceptable to the Administrative Authority.

3.702 Plans and Specifications

3.702.1 REQUIRED - Plans shall be submitted to indicate conformance with this Code and the Mechanical Code of this jurisdiction, and shall be reviewed by the department prior to issuance of the permit.

Note: Since the Fire Department is responsible for inspection for the proper maintenance of all fire protection systems in buildings, the Administrative Authority shall cooperate with the Fire Department in the discharging of its responsibility to enforce this article.

3.702.2 PLANS - The plans and specifications submitted to the department shall contain sufficient detail to evaluate the hazard and to evaluate the effectiveness of the system recommended. The details required on the hazard shall include materials involved, location and arrangement and the exposure to the hazard.

3.702.3 CALCULATIONS - Details of the fire protection system shall include design considerations, calculations and other information as required by this Code, and the Mechanical Code (Part 4 of this Code).

3.703 Fire Suppression Systems

3.703.1 TYPES - There are five main types of fire suppression systems:

(a) water sprinklers (or flood spraying);

(b) foam extinguishing systems;

(c) carbon dioxide or halogenated (suppressive gases);

(d) dry chemical extinguishing systems; and

(e) hose reel systems

See 3.704 for selection of the type of system most suitable for the building and/or occupancy involved.

3.703.2 WHERE REQUIRED - Fire suppression systems shall be installed and maintained in full operating condition, as specified in this Code, in the locations indicated in 3.703.3 to 3.703.20 with the exception of one and two-family dwellings.

3.703.3 ASSEMBLY (A-1) USE - In all buildings or portions thereof of A-1 (assembly, theatres) Occupancy group.

EXCEPTION: Auditoriums, foyers, lobbies, toilet rooms.

3.703.4 ASSEMBLY (A-2) USE - In all buildings or structures or portions thereof of use group (A-2) (assembly, nightclubs).

(a) when more than 450 sq. metres in area; or
(b) when more than one storey in height.

3.703.5 ASSEMBLY (A-3) USE - In all buildings or structures or portions thereof of use group A-3 (assembly) when more than 1,000 sq. metres in area.

3.703.6 STAGES IN ASSEMBLY (A) USE - Stages of any size in assembly occupancies (A) in the following locations:

(a) over the stage;

(b) open framework above stage (with no ceiling) and when side wall sprinklers with 57°C (135°F) rated heads with baffle plates are installed around the entire perimeter of the stage at points not more than 750 mm below the framework nor more than 150 mm below the baffle plate;

(c) under all fly-galleries;

(d) over the proscenium opening on the stage side

(e) under the stage, if more than 1.3 metres of headroom at any point;

(f) in all basements, cellars, workrooms, dressing rooms, storerooms and property rooms; and

(g) in toilets, lounge and smoking rooms.

3.703.7 HIGH HAZARD USE (H) - In all buildings or structures or portions thereof of use group H (High Hazard).

3.703.8 INSTITUTIONAL (I) USE - In all buildings or structures or portions thereof of use group I (Institutional).

EXCEPTIONS

(a) one storey hospitals and nursing homes with patient rooms having direct egress to grade or ground level at exterior of building.

(b) in hospitals of Type 1 construction, the automatic fire suppression system may be omitted from operating rooms, X-Ray rooms, delivery rooms, intensive care rooms and patients' sleeping rooms not exceeding 60 sq. metres in area, when each room is protected by an automatic fire alarm system connected to central enunciator panel;

(c) one storey day nurseries housing 100 children or less with each room having an exit directly to outside.

(d) I-1 (Institutional - restrained) occupancies having an occupancy load of less than six (6); and

(e) In I-1 (Institutional - restrained) occupancies, the fire suppression system shall be of a sprinkler system which may be either manual or automatic in operation.

3.703.9 MERCANTILE (M) MODERATE HAZARD STORAGE (S-1), OR FACTORY AND INDUSTRIAL (F) USES - In all buildings or structures of use groups M, S-1 and F - (Mercantile, Moderate hazard storage, or Factory and Industrial:

(a) when more than 1,200 sq. metres in area; or

(b) when more than 2,400 sq. metres in total area on all floors; or

(c) when more than three stories in height.

3.703.10 PUBLIC GARAGES - In all public garages;

(a) when more than 1,000 sq. metres in area; or

(b) when more than 750 sq. metres in area and more than one storey in height; or

(c) when more than 500 sq. metres in area and more than two stories in height; or

(d) when more than three stories in height; or

(e) when located in buildings where the upper storey or stories are designed for other uses; or

(f) when located in any storey or area that is more than 50% below ground level.

EXCEPTION: Open bus garages.

3.703.11 BUS GARAGES - In all bus garages

(a) when required by 3.703.10; or

(b) when used as passenger terminals for five or more buses; or

(c) when used for storage or loading of five or more buses.

3.703.12 UNLIMITED AREA BUILDINGS - In Unlimited Area Buildings as required by Part 3, Section 2, General Building Limitation - of this Code, (one storey buildings in other than frame construction, buildings of assembly - use group A-3 Business - B; Factory and Industrial - F; Mercantile - M and Storage - S, use groups which do not exceed one storey or 25 m in height may be of unlimited area, provided that exitway facilities comply with the relevant code).

EXCEPTIONS: Buildings as above but complying with type 2 or Type 3-A construction, used exclusively for storage of non-combustible material, not packed or crated in combustible material, or as specifically exempted for special industrial uses.

3.703.13 STORAGE AND WORKSHOP AREAS - In all portions of use groups A (Assembly), B (Business), I (Institutional), or R-1 and R-2 (Residential, Hotels and multi-family) occupied for storage, workshop or similar purposes.

EXCEPTIONS: Individual storage or workshop areas located entirely within a dwelling unit, or workshop areas not used for commercial purposes entirely located in the dwelling unit.

3.703.14 CELLAR OR BASEMENT - In every cellar or basement where there is less than two sq. metres of openings to every 15 metres of exterior wall entirely above ground level and on two walls of such area or storey. To qualify as part of the two square metres of area required, all openings shall have a minimum dimension (height or width) of 550 millimetres. Such openings shall be unobstructed to allow firefighting and rescue operations from the exterior.

EXCEPTIONS: Irrespective of the above (two sq. metres per 15 metres of wall), if the area of any cellar or basement exceeds 250 square metres, a fire suppression system is required.
For purposes of this requirement (3.703.14) an opening in an exterior wall may be:

(i) doors or access panel(s); or

(ii) windows, provided they have a breakable glass area of not less than 550 millimetres in the least clear dimension. All such openings shall be unobstructed to allow fire fighting and/or rescue operations from the exterior.

3.703.15 PAINTING ROOMS - In spray painting rooms or in shops where painting, brushing, dipping or mixing is regularly conducted using flammable materials.

3.703.16 LAUNDRY ROOMS, RUBBISH STORAGE ROOMS AND CHUTES -In rooms or areas used for incineration, rubbish/trash, laundry collection or similar uses. At alternate floor levels and at the top of all chutes used in conjunction with these rooms or areas.

3.703.17 BOILER ROOMS - In boiler rooms, furnace rooms, or rooms having similar uses.

3.703.18 UNENCLOSED VERTICAL OPENINGS - In all unenclosed vertical openings, between floors in a building eg. stair and light walls. (See Part 3, Section 4 sub-section 3.403 Light Ventilation and Sound Transmission).

3.703.19 RANGE HOODS - All large range hoods in hotels, hospitals, and other institutions where Local Authorities consider the risk is high and a fire suppression system must be installed, such fire suppression system shall be either a carbon dioxide (CO_2), or a halogenated gas, or a dry chemical system - (not water), and where natural or Liquid Petroleum gas is used as a fuel, a manual reset safety valve shall be installed on the gas service line to prevent fuel from flowing into the burner(s) in the event of activation of the suppression (extinguishing) system.

3.703.20 ALTERNATE PROTECTION - In special use areas of buildings an automatic fire alarm system may be installed in lieu of a fire suppression system when approved by the Fire Authority, and the authority issuing the permit, and when such fire suppression system installation would prove detrimental or dangerous to the specific use of the occupancy.

3.703.21 TELEPHONE CENTRAL OFFICE EQUIPMENT BUILDINGS AND TELEPHONE EXCHANGES - Within telephone central office equipment buildings and telephone exchanges, automatic fire sprinklers may be omitted in the following rooms or areas when such rooms or areas are protected with an approved automatic fire alarm system.

(a) generator and transformer rooms; and

(b) communication equipment areas when such areas are separated from the remainder of the building by a one-hour fire-resistance rated wall and two-hour fire-resistance rated floor-ceiling assemblies, and are used exclusively for such equipment.

3.704 Selection of Suppression System

3.704.1 GENERAL - To guide the Administrative Authority with the proper type of fixed fire protection system and the extinguishing agent for each type of hazard, fires may be classified as follows:

CLASS A: Fires involving organic solids producing glowing ember and ordinary combustible materials (such as wood, cloth, paper, rubber and many plastics), requiring the heat-absorbing (cooling) effects of water, water solutions, or the coating effects of certain dry chemicals which retard combustion.

CLASS B: Fires involving flammable or combustible liquids, flammable gases, greases and similar materials where extinguishment is most readily secured by excluding air (oxygen), inhibiting the release of combustible vapors or interrupting the combustion chain reaction.

CLASS C: Fires involving energized electrical equipment where safety to the operator requires the use of electricity non-conductive extinguishing agents; and

NOTE: Electrical fires should not be fought with portable class A or B extinguishers or with hand-held solid stream nozzle(s). However, fixed water spray systems may be used to fight fires in energized electrical systems.

CLASS D: Metals (e.g. Aluminium, Magnesium, etc.)

3.704.2 SPECIAL HAZARDS - In rooms of buildings containing combustibles, such as aluminium powder, calcium carbide, calcium phosphide, metallic sodium or potassium, quicklime, magnesium powder, or sodium peroxide, which are incompatible with the use of water, as an exitinguishing agent, other extinguishing agents, shall be used.

3.704.3 TESTS - Where a fire suppression system is required in this Code, Table 3.704 may be used by the Administrative Authority to determine the type of suppression system suitable for the hazard involved, if not otherwise specified in this Code.

TABLE 3.704
(Sub-sections 3.705 to 3.711)
GUIDE FOR SUPPRESSION SYSTEM SELECTION

Hazard	Water Sprinklers or spray	Foam	Carbon Dioxide or halogenated	Dry Chemical
Class A fire potential	x	x	x	x
Class B fire potential	x	x	x	x
Class C fire potential	x		x	
SPECIAL FIRE HAZARD AREAS*				
Aircraft hangers	x	x	x	x
Alcohol storage	x	x	x	x
Ammunition loading	x			
Ammunition magazines	x			
Asphalt impregnating	x	x		
Battery rooms			x	
Carburettor overhaul shops	x	x	x	x
Cleaning plant equipment	x	x	x	x
Computer rooms	x		x	
Drying ovens	x		x	x
Engine test cells	x	x	x	
Escalator, stair wells	x			
Explosives: manufacturing, storage	x			
Flammable liquid storage	x	x	x	
Flammable solids storage	x			
Fuel oil storage	x	x		
Hanger floors	x	x		
Hydraulic oil, lubricating oil	x		x	
Hydro-turbine generators	x		x	
Jet engine test cells	x	x	x	
Library stacks	x		x	
Lignite storage and handling	x			
Liquefied petroleum gas storage	x			
Oil quenching bath	x	x	x	x
Paints: manufacturing, storage	x	x	x	x
Paint spray booths	x		x	x
Petrochemical storage	x	x	x	

Table 3.704 (cont'd)

Hazard	Water Sprinklers or spray	Foam	Carbon Dioxide or halogenated	Dry Chemical
Petroleum Testing laboratories	x	x	x	
Printing presses	x		x	
Range Hoods	x		x	x
Record Vaults	x		x	
Rubber mixing and heat treating	x			
Service station (inside buildings)	x		x	
Shipboard storage	x		x	
Solvent cleaning tanks		x	x	x
Solvent thinned coatings		x	x	x
Switchgear rooms			x	
Transformers, circuit breakers (outdoors)	x			
Transformers, circuit breakers (indoors)	x		x	
Turbine lubricating oil	x	x	x	x
Vegetable oil, solvent extraction	x	x		

Within buildings or areas, so classified, as to require a suppression system.

3.704.4 INSTALLATION - Fixed fire suppression systems shall be of an approved type designed and installed in accordance with the requirements of this Code.

3.704.5 TESTS - All tests required by this Code shall be conducted at the expense of the owner or his representative.

3.705 Water Sprinkler Systems

3.705.1 GENERAL - Water sprinkler extinguishing systems shall be of an approved type and installed in accordance with the provision of this Code.

3.705.2 OCCUPANCY SPRINKLER SYSTEMS - Within a building of mixed occupancies and where an occupancy is required by this code to be sprinklered with more than 20 sprinklers, the area shall be enclosed by construction assemblies (fire-resistant walls, floors and ceilings for a two-hour fire rating) as required by this Code and equipped with a complete sprinkler system.

3.705.3 DESIGN - The details on the system supplied with the plans and specifications shall include all information and the calculations on the sprinkler spacing and arrangement with water supply and discharge requirements, size and equivalent lengths of pipe and fillings and water supply source. Sufficient information shall be included to identify the apparatus and devices used.

3.705.4 ACTUATION - Water sprinkler extinguishing systems shall be automatically actuated unless specifically provided in this code (spray heads actuated by smoke or heat).

3.705.5 SPRINKLER ALARMS - Approved audible or visual alarm devices shall be connected to every water sprinkler system and such alarm device shall be located in an approved location.

EXCEPTION: Alarm and alarm attachments shall not be required for limited area sprinkler systems (see sub-section 3.706).

3.705.6 ADDITIONAL ALARMS - At least one additional audible or visual alarm device shall be installed within the building.

3.705.7 WATER CONTROL VALVE TAGS - Identification tags shall be provided in accordance with relevant standards.

3.705.8 SPRINKLER RISER - The sprinkler system riser(s) may also serve as the wet standpipe riser(s) in buildings required to have both systems, or in buildings having both systems (see sub-section 3.712 - Standpipe Systems).

3.705.9 TESTS - A completed system shall be tested hydrostatically for two hours without visible leakage at not less than 1.4 MPa or at 0.35 MPa in excess of the maximum static pressure when such maximum static pressure is in excess of 1.0 MPa.

3.706 Limited Area Sprinkler Systems

3.706.1 GENERAL - A limited area sprinkler system shall be of an approved type and installed in accordance with the provisions of this Code and all relative standards.

3.706.2 INSTALLATION - Where the provisions of this Code require a limited number of sprinklers, a limited area sprinkler system shall be installed to comply with these requirements.

3.706.3 DESIGN - The design shall comply with that required by clause 3.705.3 in all respects.

3.706.4 ACTUATION - A limited area sprinkler extinguishing system shall be automatically activated (see clause 3.705.4).

3.706.5 SPRINKLER ALARMS - Alarms and alarm attachments are not required.

3.706.6 WATER SUPPLY - Limited area sprinklers may be supplied from the domestic water system provided that the system is designed to adequately support the design flow of the largest number of sprinklers in any one of the enclosed areas. When supplied by the domestic water system, the maximum number of sprinklers in any one enclosed room or area shall not exceed twenty heads (sprinklers) which must totally protect the room or area. Minimum capacity of water shall be 22,500 litres.

3.706.7 FIRE DEPARTMENT CONNECTIONS - A Fire Department connection is not required for limited area systems supplied from the domestic water system.

3.706.8 STANDPIPE CONNECTION - The water supply for the limited area system shall be from the building standpipe system (See sub-section 3.712 - Standpipe Systems).

3.706.9 CROSS CONNECTION - A limited area sprinkler system shall be supplied individually from the domestic water system or from the standpipe system. There shall be no cross connection between the domestic and the standpipe system.

3.706.10 USE - Limited area sprinklers shall be used only in rooms or areas enclosed with construction assemblies (surrounding walls being of fireproof construction and with no less than a two-hour rating) as required by this Code.

3.707 Water Spray Fixed Systems (Flood Spray Systems)

3.707.1 GENERAL - Water spray extinguishing systems shall be of an approved type and installed in accordance with the provisions of this Code and other relevant standards such as NFPA 15 USA and Fire Offices Rules 402 20 29th Edition.

3.707.2 NAME - Water spray fixed systems are also known in some Commonwealth countries as "Flood Spray Systems" and as opposed to sprinkler systems produce much more cooling, damping, and flooding effects by virtue of heavier pipework and "spray" heads needed to produce this greater volume of water.

3.707.3 DESIGN - The design shall comply in all respects with that required by clause 3.705.3.

3.707.4 ACTUATION - Water spray extinguishing systems shall be of the automatically actuated types with supplementary auxiliary manual tripping capability.

3.707.5 TESTS - All new system piping shall be hydrostatically tested in accordance with the provisions of the standard requirements specified in clause 3.705.9.

3.708 Foam Extinguishing Systems

3.708.1 GENERAL - Foam extinguishing systems shall be of an approved type and installed in accordance with the provisions of this Code and/or any relevant codes (such as NFPA 11, 11A and 16).

3.708.2 DESIGN - The detail on the system supplied with the plans and specifications shall include complete computations showing pressure drop in all system piping, friction loss calculations on liquid lines, and a detailed layout of the entire hazard to be protected. All characteristics of the foam proportions and foam makers, as determined by tests, shall be supplied by the manufacturer (through the designers) to the client or department to permit checking of the adequacy of the hydraulics of the proposed protection.

3.708.3 ACTUATION - A foam extinguishing system shall be automatically actuated but with supplementary auxiliary manual tripping capability.

3.708.4 TESTS - All piping, except that which handles expanded foam, shall be subjected to a two-hour hydrostatic pressure test at 1.4 MPa or 1.0 MPa in excess of the maximum pressure anticipated, whichever is the greater, without leakage. The system shall also be subjected to a flow test to show that the hazard is fully protected in conformance with the design and to determine flow pressures, actual discharge capacity, foam quality, consumption rate of foam-producing materials, manpower requirements and other operating characteristics.

3.709 **Carbon Dioxide Extinguishing Systems**

3.709.1 GENERAL - CO_2 systems shall be of an approved type and installed in accordance with this Code and/or other relevant codes.

3.709.2 DESIGN - The detail on the system supplied with the plans and specifications shall include information and calculations on the amount of carbon dioxide; the location and flow rate of each nozzle (including orifice area) and the size of, and location of, the CO_2 storage facility. Information shall also be submitted about the location and function of the detection devices, operating devices, auxiliary equipment, and any electrical circuits if used. Any or all special features of the system must be adequately explained.

3.709.3 ACTUATION - Carbon dioxide extinguishing systems shall be automatically actuated with supplementary auxiliary manual tripping capability.

3.709.4 SAFETY REQUIREMENTS - In any proposed use of CO_2, where there is any possibility that persons may be trapped in, or enter into atmospheres made hazardous by gas discharge, warning signs, discharge alarms, and breathing apparatus shall be provided to ensure prompt evacuation of, or to prevent entry to, such atmospheres, or to provide rescue of any trapped persons.

3.709.5 TESTS - A completed system shall be tested for tightness up to the selector valve and for continuity of piping with labelling of devices with proper designations and instructions checked. Operational tests shall be conducted on all parts of the system (except cylinder valves in multi-cylinder high pressure systems).

3.709.6 RANGE HOODS - In addition to the above requirements, range hood CO_2 systems shall conform to the following listed requirements.

(a) where more than one hood (in the same general area) is to be served, each hood shall have a separate manual station (actuator) and a separate CO_2 supply;

(b) total CO_2 requirements shall be calculated first from the manufacturers' tables/charts;

(c) upon activation of CO_2 system, the fan(s) shall cease to operate and the supply valve shall shut the pilot and burner(s) off;

(d) duct systems from range hoods shall not be equipped with fire dampers unless specifically approved for such use, or are required as part of an approved extinguishing system or an approved fan by-pass system.

(e) CO_2 bottles shall be located at least five metres from the range or range hood. The temperature in the storage area shall not exceed 50 degrees Celsius or be less than 0 degrees Celsius; and

(f) an electric warning light of not less than 10 watts shall be provided on the CO_2 bottle or system which will automatically light up when the bottle or system is depleted. The light shall be of a distinctive (preferably red) colour and shall be located in a conspicuous location.

3.710 Halogenated (Gas) Fire Extinguishing Systems

3.710.1 GENERAL -

(a) the halogens produce exactly the same effect in smoking fires (keeping oxygen out) as does carbon dioxide;

(b) all requirements given above in clauses 3.709.1 - 3.709.6 apply to this type of fire protection system; design; actuation; safety requirements and tests; and

(c) the reasons halogenated gases are used in lieu of carbon dioxide are:

(i) that they are more available in some areas or less expensive to produce; or

(ii) in the rare event that CO_2 would have some adverse chemical effect on the stored material where a fire is likely to occur.

3.711 Dry Chemical Extinguishing Systems

3.711.1 GENERAL - Dry chemical extinguishing systems shall be of an approved type, and installed in accordance with the provisions of this Code and/or other relevant codes (such as the NFPA 17).

3.711.2 DESIGN - The details on the system supplied with the plans and specifications shall include sufficient information and calculations on the amount of dry chemical; the size, the length and arrangement of connected piping, or piping and hose; description and location of nozzles so that the adequacy of the system can be determined. Information shall be submitted pertaining to the location and functioning of detection devices, operating devices, auxiliary equipment and electrical circuitry, if used. Sufficient information shall be indicated to identify properly the apparatus and devices used. Any special features shall be adequately explained.

3.711.3 ACTUATION - A dry chemical extinguishing system shall be automatically actuated with supplementary auxiliary manual tripping capability.

3.711.4 SAFETY REQUIREMENTS - Where there is any possibility that personnel may be exposed to a dry chemical discharge, warning signs, discharge alarms and breathing apparatus shall be provided to ensure prompt evacuation of such locations and also to provide means for rescue of any trapped personnel.

3.711.5 TESTS - A completed system shall be tested by a discharge of expellant gas through the piping and nozzles. Observations for serious gas leakage and for continuity of piping with free unobstructed flow of expellant gas through all nozzles. The labelling of devices with proper designations and instructions should be checked. After testing, all piping and nozzles are to be blown clean, using compressed air or nitrogen and the system properly charged and placed in the normal "set" condition. All tests are to be conducted as indicated in the above standard.

3.711.6 RANGE HOODS - In addition to the above requirements, range hood dry chemical systems shall conform to the following:

(a) dry chemical systems shall bear the label of a nationally recognized testing or inpsection agency and shall be installed in accordance with their recommendations and shall be approved by the Local Authority and the Fire Department;

(b) the size of the hood(s) and duct(s) covered by a single system shall not exceed the agency's recommendations;

(c) the dry chemical agent used shall be non-toxic;

(d) multiple hoods may be protected by a common system if in conformance with a report of the nationally recognized testing or inspection agency;

(e) each duct system shall constitute an individual system serving only exhaust heads on one floor;

(f) dry chemical bottles shall be located at least five metres from the range or range hood, or as approved by the Local Authority and the Fire Department; and

(g) an approved fire extinguisher shall be provided and located not more than five metres and not less than three metres from the hazard.

3.712 Standpipe Systems

3.712.1 GENERAL - All buildings shall be equipped with 65 mm or larger standpipes and shall be made to comply with the requirements of this section.

3.712.2 WHERE REQUIRED - Standpipes shall be installed and maintained in full operating conditions, as specified in this article and relevant standards (e.g. NFPA 14 - 76 USA) and in the locations described in Clauses 3.712.3 to 3.712.6.

3.712.3 OCCUPANCY GROUPS - A-1, A-2 and A-3 (ASSEMBLY) - In buildings two or more stories in height, of occupancy group A-1, A-2 or A-3 (Assembly) with an occupancy load of more than three hundred persons.

3.712.4 THREE STOREY BUILDINGS - In buildings three stories in height when:

(a) occupancy groups B (Business), F (Factory and Industrial), M (Mercantile) or S-1 (Moderate Hazard Storage) more than 300 sq. metres in area per floor; or

(b) of occupancy groups A (Assembly), I (Institutional), or R-1 (Residential Hotels); or

(c) of any occupancy group more than one thousand square metres in area per floor.

3.712.5 FOUR STOREY BUILDINGS - In buildings four stories or more in height, regardless of the area per floor.

3.712.6 PUBLIC GARAGES - In all public garages:

(a) when more than one thousand sq. metres in area; or

(b) when more than seven hundred and fifty sq. metres in area and more than one storey in height; or

(c) when more than five hundred sq. metres in area and more than two stories in height; or

(d) when more than three stories in height; or

(e) when located in buildings where the upper stories are designed for other uses; or

(f) when located in any storey or area that is more than fifty per cent below ground level.

3.712.7 SIZES - Standpipes (rising mains) shall extend from the lowest portion of the building to a height of one metre above the finished floor of the topmost storey and shall have a minimum diameter as described in Table 3.712.

Table 3.712
BUILDING HEIGHT AND STANDPIPE SIZE

Maximum Building Height	Minimum Standpipe Size ***
3 stories or 12 metres	65 millimetres
4 stories or 15 metres	65 millimetres
5 stories or 20 metres*	100 millimetres
6 stories or 24 metres	100 millimetres
Over 24 metres in height**	150 millimetres

* In all buildings of five stories or more or 20 metres in height or more, at least one standpipe shall extend through the roof and terminate in a two-way, 65 mm hose connection.

** A standpipe of 150 mm diameter is satisfactory up to a height of 100 metres; should a rising main or standpipe be required at any greater height than this, the engineer must produce hydraulic calculations for the Controlling/Approving Authority.

*** In sprinklered buildings only, the minimum standpipe diameter may be based on hydraulic calculations (which must be submitted to Controlling/Approving Authority).

3.712.8 NUMBER OF RISERS - The number of standpipe risers shall be such that all parts of every floor can be reached by a 10 metre hose stream from a nozzle attached to not more than thirty metres of hos connected to a riser outlet. In those buildings equpped with an interior enclosed smoke-proofed vestibule, at least one standpipe hose connection shall be located in that vestibule.

3.712.9 COMBINATION - The standpipe system riser(s) may also serve as the water sprinkler system riser(s) in buildings required to have both systems, or in buildings having both systems. A control valve shall be installed in each sprinkler system or standpipe to allow the system to remain operational.

3.712.10 OUTLETS

3.712.11 HOSE CONNECTIONS - At each floor level and not more than one and one-half metres (1.5 m) above the floor, there shall be connected to each standpipe one sixty-five millimetre hose connection and a twenty millimetre hose connection.

NOTE: The smaller hose connection may be 20 mm, 25 mm, or 30 mm, depending on the standard hose-reel hose diameter used in that particular area or country. The decision on actual size shall be made by the local governing authority in consultation with the Local Fire Authority. In any case, the size of connection with valves and threads must conform to the local Fire Department standard. Each 20 mm (or 25 mm or 30 mm - see above) hose connection shall be equipped with not more than thirty metres of approved fire hose with an approved variable fog nozzle and couplings and hung in an approved turning rack and/or cabinet with reel. Hose provided for rack and/or cabinet use shall be of rubber or other approved material and designed to be pulled out to its full length for use with the minimum effort. In many countries these come in standard units known as "hose-reels".

EXCEPTION: In fully sprinklered buildings the smaller (20, 25, or 30 mm) hose connection and hose-reel with or without cabinet is not required.

3.712.12 ROOF HYDRANT - Where standpipes extend through the roof of a building, an approved hydrant or manifold shall be provided. The main control valve on a roof hydrant or manifold shall be located in an area as close to the roof access as practical, and plainly marked.

3.712.13 MATERIAL - All standpipes shall be of galvanized wrought iron or other pipe fittings, and valves shall be of extra heavy pattern/construction when the working pressure will exceed 0.5 MPa.

3.712.14 CAPACITY - Each standpipe shall be sized for a minimum flow of 2250 litres per minute (approximately 500 gallons per minute). Where only one standpipe is required, its supply piping shall be sized for this same volume per minute (2250 litres per minute) for the first standpipe plus 1125 litres per minute for each additional standpipe, the total not to exceed 11250 litres per minute (or 2,500 gallons per minute). The supply shall be sufficient to maintain a residual pressure of 0.45 MPa at the topmost outlet of each standpipe with 2,250 litres per minute flowing.

3.713 Fire-fighting Provisions for Buildings Under Construction (or Demolition)

3.713.1 GENERAL

(a) standpipes required by this section may be temporary or permanent in nature, provided, however, that such standpipes and the water supply remains in the service until completion of the works;

(b) all buildings under construction (and when required by Local and/or Fire Authority, for buildings in the course of demolition) shall have a water supply suitable for fighting incipient fires; the water supply shall be carried on to the site(s) before any actual building work in commenced - before offices or builders sheds are erected or before any combustible materials are delivered to the site;

(c) for buildings of up to 250 square metres in area and of one storey, one domestic sized supply pipe and one standard hose tap connection is acceptable, but it must remain on site and alive for the duration of the contract; and

(d) all single storey buildings of greater area than 250 square metres, and all buildings of two or more stories in height shall comply with the requirements set out below.

3.713.2 STANDPIPES - Every building under construction - five or more stories in height above ground level shall be equipped with one or more standpipes of at least one hundred millimetres in diameter. A sufficient number of standpipes with hose(s) shall be provided so that every portion of the building can be reached with thirty metres of hose and a ten metre hose stream.

3.713.3 MATERIALS - All standpipes shall be constructed of approved pressure pipes and all pipe fittings and valves shall be extra heavy pattern/construction when the working pressure exceeds one and one-fifth megapascals (1.20 MPa).

3.713.4 HEIGHT - The standpipe systems, whether temporary or permanent, shall be carried out with each floor and shall be installed and ready for use as each floor progresses. Standpipes shall be not less than one floor below the highest forms or staging.

EXCEPTION: In multi-storey buildings being constructed of steel frame or of reinforced concrete where the forms (boxing) are completely made of non-combustible materials (such as steel sheet, asbestos sheets or similar) it is only necessary to comply with 3.713.1 (b) and (c) until such time as any combustible material is being used.

3.713.5 FIRE DEPARTMENT CONNECTIONS - At the street level there shall be provided for each temporary or permanent standpipe installation, one or more two-way Fire Department hose connections. These Fire Department connections shall be permanently marked and readily and easily accessible at all times (see sub-section 3.714).

3.713.6 OUTLETS - At each floor level and on each standpipe, there shall be provided one sixty-five millimetre hose outlet and one sixty-five millimetre hose valve with cap and chain. Also at each floor level and on each standpipe there shall be provided a smaller hose outlet (of 20 mm, 25 mm or 30 mm) depending on the standard size of hose-reels used in the district or area (see under 3.712.11 "Note" above). The size to be used will be decided by the Local Fire Authority. To this smaller hose outlet shall be attached one thirty metre length of approved hose reel or rack (one hose-reel). Both outlets shall be located not more than one and one-half metres (1.5 m) above floor level.

3.713.7 BUILDINGS UNDER DEMOLITION - Where a building is being demolished and a standpipe (or standpipes) exist(s) within such a building, such standpipe shall be maintained in an operable condition so as to be available for use by the Fire Department. Such standpipe shall be demolished with the building, but the standpipe shall not be more than one floor below to where demolition has progressed.

3.714 Fire Department Connections

3.714.1 REQUIRED - All water sprinkler and standpipe systems shall be provided with at least one two-way Fire Department connection. Each inlet of the Fire Department connection shall be at least sixty-five millimetres in diameter. The pipe from the standpipe system to the Fire Department connection shall not be smaller than one hundred millimetres. The pipe from the water sprinkler systems to the Fire Department connectin shall not be smaller than one hundred millimetres in diameter. Single Fire Department connections may be installed when approved by that department.

EXCEPTIONS: A Fire Department connection shall not be required for limited area sprinkler system. (See clauses 3.706.1 to 3.706.8).

3.714.2 CONNECTIONS - Fire Department connections shall be arranged in such a manner, that the use of any one water sprinkler connection will serve all the sprinklers, and the use of any one standpipe connection will serve all the standpipes within the building.

3.714.3 LOCATION - Fire Department connections shall be located and be visible on a street front or in a location approved by the appropriate department. Such connections shall be located so that immediate access can be made by the Fire Department. Obstructions such as fences, bushes, trees, walls or any other similar objects shall not be permitted for new or existing installations.

3.714.4 HEIGHTS - Fire Department connections shall not be less than one-half metre (0.5 m) and not more than one metre in elevation, measured from ground level to the centre line of the inlets.

3.714.5 PROJECTION - Where the Fire Department connection would project beyond the property line or into the/a public way, a flush type fire department connection shall be provided.

3.714.6 HOSE THREADS - Hose threads in the Fire Department connection shall be uniform with that used by the local Fire Department.

3.714.7 FITTINGS - Fire Department inlet connections shall be fitted with check-valves, or ball-drip valves and caps and chains.

3.714.8 SIGNS - A metal sign with raised letters at least twenty-five millimetres in height shall be mounted on all Fire Department connections serving sprinklers and/or standpipes. Such signs shall read "Automatic Sprinklers" and/or "Standpipe".

3.715 Water Supply

3.715.1 REQUIRED - All fire suppression and standpipes systems shall be provided with at least one automatic supply of water of adequate pressure, capacity and reliability to perform the function intended.

3.715.2 COMBINATION SPRINKLER/STANDPIPE WATER SUPPLY - Where both sprinklers and standpipes are installed, they may have a common fire water service as their combined course of supply. The connection shall not be made to any water main of less than one hundred millimetres in diameter. In sprinklered buildings with combined standpipes, the water supply shall be adequate for the spinkler system or the standpipe system, whichever is the greater.

3.715.3 COMBINATION SPRINKLER/DOMESTIC WATER SUPPLY - A spinkler system may be connected to the domestic water supply system as allowed by this Code, provided that the supply system is of adequate pressure, capacity and size for the simultaneous operation of the water sprinkler system and the domestic water needs. A check valve shall be installed in the water sprinkler supply line to prevent contamination of the domestic water.

3.715.4 SIZE - The water supply for fire suppression systems shall be sized in an approved manner in accordance with this Code and all relevant standards.

3.715.5 STANDPIPES - WATER SUPPLY - Standpipes shall be connected to a street water main with a fire water service at least equal to the size of the largest standpipe within the building, or shall be hydraulically calculated to satisfy the total demand. The size of the water service at the base of the standpipe risers, shall be at least the size of the largest standpipe.

3.715.6 INTERCONNECTION - The required water supply shall be connected to the base of each standpipe. Where more than one standpipe is required, all standpipes shall be interconnected at their base and an approved indicating valve shall be installed at the base of each standpipe so as to permit individual risers to be taken out of service, if damaged or broken, without interrupting the water supply to other risers.

3.716 Yard Hydrants

3.716.1 FIRE HYDRANTS - Fire hydrants installed on private property shall be located and installed as directed by the Fire Department. Hydrants shall conform to the standards of the Administrative Authority and the Fire Department. Fire hydrants shall not be installed on a water main of less than one hundred and fifty millimetres in diameter.

3.717 Automatic Fire Alarm Systems

3.717.1 PLANS AND SPECIFICATIONS - Where required by this Code, the plans and specifications for the automatic fire alarm systems shall show location and number of all sending stations and signals, with specifications of the type, construction and operation of the system including all automatic detection devices. Installation of all equipment shall conform to the requirements of this Code and the applicable standards thereto.

3.717.2 APPROVAL - The automatic fire alarm system shall be approved for the particular application and shall only be used for detection and signalling in the event of fire. The automatic detecting devices shall be approved smoke (or heat) detectors.

3.717.3 WHERE REQUIRED - An automatic fire alarm system shall be installed and maintained in full operating condition in the locations described in clauses 3.717.4 to 3.717.6.

3.717.4 INSTITUTIONAL (I) USE - In all buildings of occupancy group I (Institutional).

3.717.5 RESIDENTIAL (R) USE - In all buildings of occupancy group R-1 (Residential hotels).

EXCEPTION: Buildings over six stories or twenty-two metres in height equipped with an automatic fire suppression system.

3.717.6 RESIDENTIAL USE - In each guest room, suite, or sleeping area of occupancy groups R-1 (Residential, hotel, motel, lodging house, boarding house and dormitory) or dwelling unit within buildings of occupancy R-2 (Residential, multi-family). Each unit shall be provided with a minimum of one approved smoke or heat detector installed in a manner or location approved by the governing authority. When actuated, the detector shall provide an alarm suitable to warn the occupants within the individual unit (see sub-section 3.718). In buildings having basements or cellars an additional detector shall be installed in the basement or cellar in a location approved by the local authority having jurisdiction. Smoke or heat detectors required by this Article shall comply with the relative standard.

3.717.7 SPRINKLERED BUILDINGS EXCEPTION - Buildings or portions thereof equipped with an automatic fire suppression system are not required to be equipped with an automatic fire alarm system, but are required to be equipped with a manual fire alarm system conforming to sub-section 3.718.

3.717.8 MANUAL STATIONS - A manual fire alarm system conforming to the requirements of sub-section 3.718 shall be installed in conjunction with an automatic fire alarm system.

EXCEPTION: Automatic fire alarm systems for occupancy group R-2 as required by 3.717.6.

3.717.9 DISTANCES - Approved fire-detecting devices shall be installed not to exceed the lineal allowances or areas specified based on the generally accepted test standards under which they were tested and approved.

3.717.10 NOT MANDATORY - In special occupancy buildings and structures or parts thereof, an automatic fire alarm system may be installed in lieu of an automatic fire suppression system when approved by the governing body and the fire department, when such installation would be detrimental or dangerous to the specific use and occupancy (see clause 3.703.20).

3.717.11 POWER SUPPLY - The power for the automatic fire alarm system shall be provided from an emergency electrical system (usually from batteries with trickle-charger off mains).

EXCEPTION: Automatic fire alarm systems for occupancy group R-2 as required by clause 3.717.6.

3.717.12 REQUIREMENTS - All automatic fire alarm systems shall be of the closed circuit type and shall be electrically and/or mechanically supervised. In addition, such systems shall comply with clauses 3.717.13 to 3.717.15.

3.717.13 WIRING - All wiring shall conform to the requirements of NFPA (National Fire Protection Association of USA) and relevant local standards.

3.717.14 AUDIBLE ALARMS - Audible alarms of an approved type shall be provided. The operation of any detection device shall cause all audible and/or visual alarms to operate. Visual and audible alarms shall be provided in occupancies housing the hard of hearing. Alarm sounding devices shall be approved and shall provide a distinctive tone and shall not be used for any other purpose than that of a fire alarm. They shall be located so as to be effectively heard above all other sounds by all the occupants in every occcupied space within the building.

3.717.15 ZONES - Each floor shall be zoned separately. If the floor area exceeds two thousand square metres, additional zoning shall be provided. The length of any zone shall not exceed sixty metres in any direction. Zoning indicator panels and controls shall be located as approved by the governing body and the Fire Department. Annunciators shall lock in until the system is reset.

3.717.16 TESTS - Upon completion of a fire alarm system, the installation shall be subjected to a performance test to demonstrate its efficiency of operation. Also, all connections and wiring, with signal devices disconnected, shall develop an insulation resistance of not less than one megohm.

3.718 **Manual Fire Alarm Systems**

3.718.1 PLANS AND SPECIFICATIONS - As required by this Code, plans and specifications for a manual fire alarm system shall show the location and number of all sending stations and signals with specifications of type, construction and operation of the system. Installation of all equipment shall conform to the requirements of this Code and any other applicable standards.

3.718.2 APPROVAL - The manual fire alarm system shall be installed and maintained in full operating condition in the locations described on the next page:

(a) in all buildings equipped with an automatic alarm system (see clause 3.717.8);

(b) in all new and existing buildings in occupancy group A-4 (Assembly - Educational);

(c) in all buildings in occupancy Group B (Business) when of three stories or more in height; and

(d) buildings in Residential (Multi-family) R-2 occupancy when four or more stories in height.

EXCEPTIONS: for (1) Occupancy group R-2 buildings (hotel, motel, etc);

for (2) Certain areas in churches and other religious buildings such as sanctuaries, etc; and

for (3) Buildings in Occupancy group B that are equipped with a sprinkler or other suppression system.

3.718.3 LOCATION - Manual sending stations (alarm boxes) shall be located in each common corridor of each storey, including basements. The alarm boxes shall be located not more than two metres from each exitway and the distance between any two shall not be more than 50 metres. Where corridors are not provided, alarm boxes shall be located in such a way that no point in the building is more than 50 metres from any alarm box. If the area contains a stage, then the fire alarm shall be located adjacent to the lighting control panel, switchboard, or subboard.

3.718.4 POWER SUPPLY - The power for the fire alarm system shall be provided from an emergency electrical supply; (usually from batteries with a trickle-charger off mains).

3.718.5 REQUIREMENTS -

(a) fire alarm systems shall be of a closed circuit type and shall be electrically or mechanically supervised; and

(b) all wiring, alarm bells and/or signals, zones and acceptance tests shall be as required for automatic alarm systems enumerated in 3.717.13 to 3.717.15 and 3.717.16.

3.718.6 BOX HEIGHTS - The height of manual fire alarm boxes shall not be more than 1.25 metres above floor level.

3.718.7 HAND OPERATED EXTINGUISHERS (PORTABLE TYPES)

3.718.8 TYPES - The usual types of portable fire extinguishers available are:

(a) carbon dioxide (CO_2) - for small electrical fires;

(b) dry powder extinguishers - small petrol, oil, or other combustibles; and

(c) bucket pumps - where no water supply available on temporary buildings.

3.718.9 WHERE REQUIRED - Although portable fire extinguishers cannot be regarded as a fire protection system for any building, they shall be installed where required by the Fire Authority and/or building owners in buildings of any occupancy classification that;

(a) are too small for a full alarm or suppression system, but need some first aid protection; or

(b) in buildings where the early use by staff or tenants could be vital even if they were equipped with manual (or other alarm systems); or

(c) in sections or areas of buildings that do have an automatic system but have been excluded from using water sprinklers or other systems such as in operating theatres or certain storage areas; or

(d) in temporary buildings erected for fairs, carnivals and other shortterm usage.

3.718.10 TESTS - All portable units shall be regularly tested by the Fire Department or other authorised testing agencies approved by the Local Authority/Fire Authority and records kept on dates tests were carried out. (See also under sub-section 3.719 - Supervision and Testing.)

3.719 **Supervision and Testing**

3.719.1 GENERAL - All fire protection systems and all portable extinguishers must be supervised and tested by the local Fire Authority or some other agency approved by the authority to carry out this work and keep records of tests and dates.

3.719.2 FIRE SUPPRESSION SYSTEMS - The whole of electrical and mechanical operating mechanisms of all suppression systems shall be tested at not more than three-month intervals, and a record kept by the testing authority or agency of the results of tests: one copy of these results shall be forwarded or handed to the building owner/occupier who shall keep it as a record.

3.719.3 DETECTOR HEADS - Smoke and heat detector heads shall also be tested, sprinkler heads shall not be tested.

NOTE: Although this Code does not require sprinkler heads to be tested at three monthly intervals, it is recommended that over each four-year period, sections of the sprinkler systems shall be disconnected and sprinkler heads taken away and heat and pressure tested, and any which fails to meet the test requirements shall be replaced.

3.719.4 WATER SUPPLIES - All water supply valves leads, cut-offs, connections and other piping shall be checked for leaks and corrosion. Hose-reels, hydrants, and other open outlets shall be checked for water pressure and flow. Checks should also be carried out to ensure that all valves are working.

3.719.5 OTHER FIRE EXTINGUISHING MEDIA - Carbon dioxide or other gas supplies and pressures from supply sources shall be checked and recorded and dry powder containers checked for level of contents.

3.719.6 ALARM SYSTEMS - Automatic and manual alarm systems shall be checked electrically and mechanically. Battery circuits both within buildings and from building to fire station, shall be tested and faults investigated promptly and corrected.

3.719.7 LOCAL FIRE AUTHORITY'S RIGHTS - Representatives of the Local Fire Authority have the right to enter any building or compound at any time to make tests on water mains, valves, or hydrants, if they have cause to suspect that there may be blockages or that the supply has been interrupted or that valves are leaking or sticking, or for any other valid reason.

CARIBBEAN UNIFORM BUILDING CODE

PART 3
OCCUPANCY, FIRE SAFETY AND PUBLIC HEALTH REQUIREMENTS

SECTION 8

SAFETY REQUIREMENTS DURING BUILDING CONSTRUCTION AND SIGNS

PART 3

SECTION 8

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PART 3

SECTION 8

3.800 SAFETY REQUIREMENTS DURING BUILDING CONSTRUCTION AND SIGNS

3.801 Scope

3.801.1 The provisions of this section shall apply to precautions required in building/construction operations in connection with the erection, alteration, repair, removal of signs, whether they be part of the same building contract or completely independent of each other and in respect of structural, fire, and safety codes for both workmen and the general public.

3.801.2 PUBLIC HEALTH - OTHER LAWS - Nothing herein contained shall be construed to nullify any rules, regulations or statutes of state agencies governing the protection of the public or workmen from health or other hazards involved in manufacturing, mining and other processes and operations which generate toxic gases, dust or other elements dangerous to the respiratory system, eyesight or health.

3.801.3 ZONING LAW - Where more restrictive in respect to location, use, size or height of signs, the limitations of the zoning laws affecting required light and ventilation requirements (see Part 3 Section 4 of this Code) and use of land shall take precedence over the regulations of this Code.

3.801.4 APPROVED RULES - In the absence of approved rules or a relevant code or standard governing details of construction, the provision of applicable standards such as BSI or other approved shall be deemed to conform to the requirements of this Code.

3.802 Plans, Specifications and Permits

3.802.1 TEMPORARY CONSTRUCTION - Before any construction operation is started, plans and specifications shall be filed with the building authority showing the design and construction of all sidewalk sheds, truck runways, trestles, foot bridges, guard fences, signs and other similar devices required in the operation; and the approval of the Building Authority shall be secured before the commencement of any work.

3.802.2 PERMITS - All special licenses and permits for the storage of materials on sidewalks and highways for the use of water or other public facilities and for the storage and handling of explosives shall be secured from the Local Authorities having jurisdiction in the area.

3.802.3 TEMPORARY ENCROACHMENTS - Subject to the approval of the Building Authority, sidewalk sheds, underpinning and other temporary protective guards and devices may project beyond the interior and street lot lines as may be required to insure the safety of the adjoining property and the public. When necessary, the consent of the adjoining property owner shall be obtained.

3.802.4 OWNER'S CONSENT - Before any permit is granted for the erection of a sign; plans and specifications shall be filed with the Building Authority showing the dimensions, materials and required details of construction, including loads, stresses and anchorage. The applications shall be accompanied by the written consent of the owner or lessee of the premises upon which the sign is erected, if not on the applicant's own property.

3.802.5 NEW SIGNS - A new sign shall not hereafter be erected, constructed, altered or maintained except as herein provided and until after a permit has been issued by the Building Authority and the required bond (if required) shall have been filed in accordance with sub-section 3.816 below.

3.802.6 ALTERATIONS - A sign shall not be enlarged or relocated except in conformity to the provisions of this section for new signs, nor until a proper permit has been secured. The repainting or reposting of display matter, shall not be deemed an alteration; provided the conditions of the original approval and the requirements of this section are not violated.

3.802.7 IDENTIFICATION - Every sign for which a permit has been issued and hereinafter erected, constructed or maintained shall be plainly marked with the name of the person, firm or corporation owning, erecting, maintaining or operating such sign. The method and location of this identification shall appear on the plans and within the specification filed with the Building Authority.

3.803 Tests and Inspection

3.803.1 LOADING - It shall be unlawful to load any structure, temporary support, scaffolding, sidewalk bridge or sidewalk shed or any other device or construction equipment during the construction or demolition of any building or structure in excess of its safe working capacity as provided in Part 2 Section 1 of this Code for allowable loads and working stresses.

3.803.2 UNSAFE EQUIPMENT - Whenever any doubt arises as to the structural quality or strength of scaffolding plank or other construction equipment, such material shall be replaced; provided, however, the Building Authority Inspectors may accept a strength test to two and one-half (2 1/2) times the superimposed live load to which the material or structural member is to be subjected. The member shall sustain the test load without failure.

3.803.3 REMOVAL - The Building Official may order the removal of any sign that is not maintained in accordance with the provisions of this section of the Code.

3.803.4 UNSAFE CONDITIONS - When inspection of any construction operation reveals that any unsafe or illegal conditions exist, the Building Inspector/Authority shall notify the owner and direct him to take the necessary remedial measures to remove the hazard or violation.

3.803.5 NOTICE OF UNSAFE SIGNS - When any sign becomes insecure, in danger of falling, or otherwise unsafe, or if any sign shall be unlawfully installed, erected or maintained in violation of any of the provisions of this Code, the owner thereof or the person or firm maintaining same, shall upon written notice of the Building Authority, forthwith in the case of immediate danger and in any case within not more than ten (10) days, make such sign conform to the provisions of this article or shall remove it. If within ten (10) days the order is not complied with, the Building Authority may remove such sign at the expense of the owner or lessee as provided in Part 1 Section 1 (administration and enforcement) of this Code.

3.803.6 FAILURE TO COMPLY WITH ORDERS OF UNSAFE CONSTRUCTION OPERATIONS - Unless the owners so notified (3.803.4 and 3.803.5) proceeds to comply with the orders of the building official within twenty-four (24) hours, the Building Authority shall have full power to correct the unsafe conditions as provided in Part 1 Section 1. All expenses incurred in the correction of such unsafe conditions shall become a lien on the property.

3.803.7 UNSAFE CONSTRUCTION EQUIPMENT - When the strength and adequacy of any scaffold or other device or construction equipment is in doubt, or when any complaint is made, the Building Inspector/Authority shall inspect such equipment and shall prohibit its use until tested as required under 3.803.2 above or until all danger is removed.

3.803.8 MAINTENANCE OF EQUIPMENT - All construction equipment and safeguards shall be constructed, installed and maintained in a substantial manner and shall be so operated as to ensure protection to the workmen engaged thereon and to the general public. It shall be unlawful to remove or render inoperative any structural, fire-protective or sanitary safeguard or device herein required except when necessary for the actual installation and prosecution of the work.

3.804 **Existing Buildings**

3.804.1 PROTECTION - All existing and adjoining public or private property shall be protected from damage incidental to construction operations.

3.804.2 CHIMNEY, SOIL AND VENT STACKS - Whenever a new building is erected to greater or less heights than an adjoining building, the construction and extension of new or existing chimneys shall conform to the provisions of Part 4 Section 2 of this code (Chimneys, Flues, and Vent Pipes).

3.804.3 ADJOINING WALLS - The owner of the new or altered structure shall preserve all adjoining independent and party walls from damage as provided herein. He shall underpin where necessary and support the adjoining building or structure by proper foundations to comply with 3.806 (Excavations).

3.804.4 MAINTENANCE - In case an existing party wall is intended to be used by the person who causes an excavation to be made, and such party wall is in good condition and sufficient for the use of both the existing and proposed building, such person shall preserve the party wall from injury and support it by proper foundations at his own expense, so that it shall be and shall remain safe and useful as it was before the excavation was commenced. During the demolition, the party wall shall be maintained weatherproof and structurally safe by adequate bracing until such time as the permanent structural supports shall have been provided.

3.804.5 BEAM HOLES - When a structure involving a party wall is being demolished, the owner of the demolished structure shall, at his own expense, bend over all wall anchors at the beam ends of the standing wall and shall brick up all open beam holes and otherwise maintain the safety and usefulness of the wall.

3.804.6 PARTY WALL FIRE EXITWAYS (OR ESCAPES) - A party wall balcony or horizontal exit shall not be destroyed during demolition of an adjoining building until a substitute means of egress has been provided and approved by the Local Authority.

3.804.7 ADJOINING ROOFS - When a new building or demolition of an existing building is being prosecuted at a greater height, the roof, roof outlets and roof structures of adjoining buildings shall be protected against damage with adequate safeguards by the person during the work.

3.805 **Protection of the Public and Workmen**

3.805.1 GENERAL - Whenever a building or structure is erected, altered, repaired, removed or demolished, the operation shall be conducted in a safe and workmanlike manner and suitable protection for the general public and workmen employed thereon shall be provided.

3.805.2 FENCES - Every construction operation located one and one-half metres (1.5 m) or less from the street boundary line shall be enclosed with a fence not less than 2.4 metres high to prevent entry of unauthorized persons. When located more than 1.5 metres from the street lot line, a fence or other barrier shall be erected if required by the Building Authority. All fences shall be of adequate strength to resist the wind pressure as specified in Part 2 Section 1 of this Code.

3.805.3 SIDEWALK BRIDGE - Whenever the ground is excavated under the footpath, a bridge shall be constucted at least 1.2 metres wide, or a protected walkway of equal width shall be erected in the street, provided the required permit for such walkway is obtained from the Administrative Authority.

3.805.4 FULL PEDESTRIAN PROTECTION - When any building or part thereof is to be erected either on the street boundary line or within three metres (3.0 m) of such boundary, and the building is to be erected to a height of 12 metres or more, then a fully protected footpath "shed" or covered wall shall be provided for the full length of the proposed building; should such building be on a street corner, the protection shall be continued around the corner and for the full depth of the building also.

3.805.5 Similarly, when a building of greater height than 12 metres on or within 3 metres of the street boundary is to be demolished, full protection on all/both street frontages shall be provided. This pedestrian protection structure shall remain in place for the entire time work is being performed on the exterior of the building.

3.805.6 WITHIN 6 METRES OF STREET BOUNDARY LINE - When the building being demolished or erected is located within six metres of the street boundary line and is more than 12 metres in height, exterior flare fans or catch platforms shall be erected at vertical intervals of not more than two (2) stories.

3.805.7 BUILDINGS HIGHER THAN SIX STORIES - When the building being demolished or erected is more than six (6) stories or 23 metres in height, unless set back from the street boundary a distance more than one-half (1/2) its height, a sidewalk "shed" shall be provided.

3.805.8 WALKWAY - An adequately lighted walkway at least 1.2 metres wide and 2.4 metres high in the clear shall be maintained under all sidewalk "sheds" for pedestrians. Where ramps are required, they shall conform to the provisions of Part 3 Section 5 of this Code.

3.805.9 THRUST-OUT PLATFORMS - The Building Authority may approve thrust-out platforms (sometimes known as catch platforms) or other substitute protections in lieu of sidewalk sheds when deemed adequate to ensure the public safety. Thrust-out platforms shall not be used for the storage of materials.

3.805.10 WATCHMAN - Whenever a building is being demolished, erected, or altered, a watchman shall be employed to warn the general public when intermittent hazardous operations are conducted across the footpath or walkway.

3.806 Excavations

3.806.1 TEMPORARY STRUCTURE - Until permanent support has been provided, all excavations shall be safeguarded and protected by the contractor or persons carrying out the excavation works, to avoid all danger to life or limb. Where necessary or where directed by the Local Authority Engineer or Inspector such excavations shall be retained by temporary retaining walls, sheet-piling and bracing or other approved method to support the adjoining earth.

3.806.2 EXAMINATION OF ADJOINING PROPERTY - Before any excavation or demolition is undertaken, license to enter upon adjoining property for the purpose of physical examination shall be afforded by the owner and tenants of such adjoining property, both to the person undertaking such excavation or demolition and/or the Local Authority Inspector(s) or Engineer(s) prior to commencement and at reasonable periods during the progress of the work.

3.806.3 NOTICE TO LOCAL AUTHORITY - If the demolition or excavation contractor has any reason to believe that an adjoining building or other structure is unsafe, he shall forthwith report this in writing to the Local Authority. The Local Authority Engineer(s) or Building Inspector(s) shall inspect such premises, and if the building is found to be unsafe, shall order it repaired (or demolished) as provided for in Part 1 Section 1 of this Code (Administration and Enforcement).

3.806.4 RESPONSIBILITY OF ADJOINING OWNER - The contractor or person making or causing an excavation to be made shall, before starting the work, give at least one (1) week's notice in writing to the owner of such neighbouring building or structure, the safety of which may be affected. Having received consent to enter a building or premises, he shall make the necessary provisions to protect it structurally and to insure it against damage by the elements which may ensue from such excavation or demolition. If license to enter is not afforded, then the adjoining owner shall have the entire responsibility of providing both temporary and permanent support of his premises at his own expense; and for that purpose, he shall be afforded the license when necessary to enter the property where the excavation is to be made.

3.806.5 EXCAVATIONS FOR OTHER THAN CONSTRUCTION PURPOSES - Excavations made for the purpose of removing soil, earth, sand, gravel, rock or other materials shall be performed in such a manner as will prevent injury to neighbouring properties or to the street which adjoins the property where such materials are excavated, and to safeguard the general public health and welfare.

3.806.6 LOCAL BODY/LOCAL GOVERNMENT REGULATIONS - Nothing in this code shall be deemed to override/overrule any local body building regulations or by-laws in any way whatsoever.

3.806.7 MINIMUM/MAXIMUM DEPTHS OF EXCAVATIONS - The requirement of this code is that no excavation on any property shall be carried to a greater depth than one-half of one metre (0.5 m) ABOVE the depth of the existing foundation or footing of any adjoining building or structure, without a special dispensation having been granted, in writing, from the Local Building Authority.

3.806.8 Where such Local Authority has a regulation or bylaw giving either a maximum or a minimum allowable excavation depth in any locality in its jurisdiction, this shall take precedence over clause 3.806.7 above unless as stated under 3.806.7, a special written dispensation has been granted.

3.806.9 TEST HOLES/TEST BORES - When the owner of a property, or an excavation contractor on his behalf, applies to the Local Authority for a special permit (or dispensation) to excavate to a greater depth then normally allowed, he shall arrange for, and meet the costs of, any test holes or test bores required by the Local Authority. He shall employ or use a qualified and registered engineer to assess the results of such test holes or bores, and shall submit calculations, graphs and test samples of soils if required to the Local Authority along with his application to excavate.

3.806.10 RESPONSIBILITY - In the event of any subsidence of adjoining buildings or structures, or of any slips or landslides, or of any damage to drains or other underground services to adjoining properties, no blame shall be attached to any other person, whether adjoining owners, or Local Authority - the liability rests with the owner/and or his contractor who is carrying out the excavation work.

3.806.11 RETAINING WALLS - When the ground level on any property, after a demolition or excavation on such property has been carried out is more than one and one-half metres (1.5 m) either above or below the general ground level of any adjoining property, the owner of the property on which work has been, or is being, carried out shall, at his own expense, erect a properly designed and built retaining wall. Such wall shall be of a height and strength to retain either his own or the earth on the adjoining property in a stable condition. The wall shall be entirely on his own property, and shall be provided with a guard rail or fence not less than 1.2 metres in height all to the requirements of the local authority.

3.806.12 LEVELLING OF SITE - When a building has been demolished and immediate building operations have not been projected or approved, the vacant lot shall be filled if necessary, graded and maintained in conformity to any established street grades or levels. The lot shall be maintained free from the accumulation of rubbish and all other unsafe or hazardous conditions which endanger the life or health of the public; and provisions shall be made to prevent the accumulation of water or damage to any foundations on the premises or the adjoining property.

3.806.13 UTILITY CONNECTIONS - All service utility connections shall be discontinued and capped or sealed in accordance with the approved rules and the requirements of the local agency having jurisdiction in the area.

3.806.14 WASTE MATERIAL - Rubbish, spoil, or other waste material shall not be dropped or thrown outside the exterior walls of a building during demolition or erection. Wood or metal chutes shall be provided for this purpose and any material which in its removal will cause an excessive amount of dust shall be wet down to prevent the creation of a nuisance.

3.806.15 This possible dust nuisance shall also be taken into account during excavation works, and if so required by the local authority, steps shall be taken to wet down the materials, to carry out the works after hours or at night time or alleviate the nuisance in any way the Local Authority Inspectors shall require.

3.807 Storage of Material

3.807.1 GENERAL - All materials and equipment required in construction operations shall be stored and placed so as not to endanger the public, the workman or adjoining property.

3.807.2 DESIGN CAPACITY - Materials or equipment stored within the building or on sidewalks, sheds or scaffolds shall be placed so as not to overload any part of the construction beyond its design capacity, nor interfere with the safe prosecution of the work.

3.807.3 SPECIAL LOADING - Unless the construction is designed for special loading, materials stored on sidewalk or other sheds and scaffolds shall not exceed one (1) day's supply. All materials shall be piled or stacked in an orderly manner and height, to permit removal of individual pieces without endangering the stability of the pile.

3.807.4 PEDESTRIAN WALKWAYS - Materials or equipment shall not be stored on the street without a permit issued by the Local Authority. When so stored they shall not unduly interfere with vehicular traffic, or the orderly travel of pedestrians on the highways and streets. The piles shall be arranged to maintain a safe walkway not less than 1.2 metres wide, unobstructed for its full length, and adequately lighted at night and at all necessary times for the use of the public.

3.807.5 OBSTRUCTIONS - Material and equipment shall not be placed or stored so as to obstruct access to fire hydrants, standpipes, fire or police alarm boxes, utility boxes, or manholes, nor shall they be located within six metres (6.0 m) of a street intersection, or so placed as to obstruct normal observations of traffic signals or to hinder the use of street car loading platforms.

3.808 Protection - Floors and Openings

3.808.1 NON-COMBUSTIBLE FLOOR CONSTRUCTION - The arches, slabs or structural floor fillings of buildings of fireproof construction (Type 1) and non-combustible construction (Type 2) shall be installed as the building progresses.

3.808.2 COMBUSTIBLE FLOOR CONSTRUCTION - In wood joist floor construction (Types 3 and 4) when double flooring is used, the underfloor shall be laid on each storey as the building progresses; and when double floors are not used, the floors shall be planked over two (2) stories below the level where work is being performed.

3.808.3 STEEL STRUCTURAL FRAMES - In steel construction, the entire tier of iron or steel beams upon which the structural work is in progress shall be planked over with the exception of necessary hoistways and permanent openings; and the steel work shall not advance more than six (6) floors ahead of the permanent floor construction.

3.808.4 GUARD RAILS - All floor and wall openings shall be protected with substantial guard rails and toe boards in accordance with accepted engineering practice.

3.808.5 ADJOINING PROPERTY - Adjoining property shall be completely protected from any damage incidental to the building operation when the owner of the adjoining property permits free access to the building. It is encumbent on the contractor at all times to provide the necessary safeguards in accordance with clauses under 3.806 above.

3.809 Scaffolding

3.809.1 LOAD CAPACITY - Scaffolds and their components shall be capable of supporting without failure at least four (4) times the maximum intended load. All platforms and supporting elements of scaffolds shall be designed and constructed to support uniform minimum live loads in kilograms per square metre (kg/m^2) of the platform area in accordance with the classifications described in the following Table 3.809.

TABLE 3.809
SCAFFOLD LOAD CAPACITY

Classification	Service Type	Load in kg/m^2
Light duty	Carpenters*	120
	Block layers*	120
	Miscellaneous*	120
Medium duty	Bricklayers	250
	Lath and plaster	250
	Stucco	250
	Tilers	250
Heavy duty	Stonemasons	375
	Blocklayers	375

* No blocks, bricks, stone or other materials on scaffold.

3.809.2 ERECTION - Built-up, swinging, and suspended scaffolds shall be erected by competent workmen only.

3.809.3 BUILDINGS OF FIRE-RESISTANT CONSTRUCTION - All scaffolding exceeding 25 metres or seven (7) stories in height used in construction operations involving the erection, alteration or maintenance of buildings, shall be constructed of non-combustible or fire-retardant materials complying with the provisions of Part 3 Section 6 of this Code.

3.809.4 INSTITUTIONAL BUILDINGS - All scaffolding used in construction operations involving the repair or partial demolition of institutional buildings (occupancy groups I-

1 and I-2), during occupancy of the building shall be constructed of non-combustible or fire-retardant materials complying with the provisions of Part 3 Section 6 of this Code.

3.810 **Hoists (and Cranes)**

3.810.1 HOIST PROTECTION - All materials hoists shall be adequately protected; and when erected on the outside of a building over twenty-five metres (25.0 m) or seven (7) stories in height, the structure shall be built of non-combustible or approved fire-retardant materials with the exception of the loading platform(s).

3.810.2 PASSENGERS PROHIBITED - Persons shall not be permitted to ride a material hoist. Temporary elevators shall be installed when necessary, to transport workmen as provided in Part 4 Section 4 of this Code.

3.810.3 GUARDING OF CABLES - All hoisting cables and signal cords shall be guarded wherever they pass through or cross working spaces to prevent injury to persons.

3.810.4 RIGGER'S LICENSE - All persons engaged in the erection of derricks, cranes, hoists and other hoisting apparatus shall secure a license or certificate of fitness for the performance of such work from the Authorized Administrative Authority.

3.810.5 CRANES - All cranes used on a new building site, or where an existing building is being added to shall (a) be subject of an approval or permit from the Local Authority, (b) shall be erected either on their own bases, on rails, or on a portion of the building in course of erection by qualified, tested and licensed workmen (see also 3.810.4 above) (c) shall be tested before use by the Local Testing Authority (Machinery Inspectors, Safety Inspectors, Labour Department or Marine Department Inspectors) and given a certificate of fitness and warrant to operate/license to operate, a copy of which shall be lodged with the Local Building Authority and the original certificate shall be displayed in the cab or operating section of the crane (d) the certificate/license shall be for a specific period (usually 6 months and renewable) the expiry date shall be clearly marked on the certificate. It is an offence to operate a non-certificated crane. The operator should also be a certificated crane operator.

3.810.6 If the crane is of the "hammerhead" type where the load is lifted from a moving trolley on the head member, a metal plate welded or rivetted to the unit must show the maximum loads in kilograms that may be lifted from the various distances out from the fulcrum.

3.810.7 If a (raising and lowering) single boom or jib crane is employed it too must have a rivetted or welded-on metal plate with the safe loads allowable from the truck of the boom at the various angles (given in degrees from the horizontal) that the crane is permitted to lift.

3.811 Stairways and Ladders

3.811.1 TEMPORARY STAIRWAYS - When a building has been constructed to a greater height than 15 metres or four (4) stories, or when an existing building which exceeds 15 metres in height is altered, at least one (1) temporary lighted stairway shall be provided unless one (1) or more of the permanent stairways are erected as the construction progresses.

3.811.2 LADDERS - Temporary ladders, when permitted for access to floors before stairways are installed, or which are designed for other working purposes, shall extend at least one metre (1.0 m) above the floor level which they serve.

3.811.3 LIGHTING (PROTECTION) - All stairways, ladders and any other parts of buildings under demolition, erection or repair shall be adequately lighted while persons are engaged at work. (See also Means of Egress Lighting in Part 3 Section 5 of this Code).

3.812 Fire Hazards

3.812.1 GENERAL - The provisions of this code and of the fire prevention regulations shall be strictly observed to safeguard against all fire hazards attendant upon construction operations.

3.812.2 STEAM BOILERS - All temporary or permanent high pressure steam boilers shall be operated only by licensed operating engineers in accordance with the provisions of the mechanical section of this Code (Part 4 Section 1). When located within a building or within 3 metres thereof, all such boilers shall be enclosed with approved non-combustible construction.

3.812.3 STORAGE OF FLAMMABLES - Storage of gasoline for hoists, oils, paints and other highly flammable materials shall be permitted only as specified in Part 3 Section 3 of this Code and when stored in approved safety containers. The storage of larger quantities may be approved by the local fire and/or explosives authority when stored in separate compartments or enclosures of approved non-combustible construction.

3.812.4 FLAME CUTTING AND WELDING - The use of oxyacetylene torches for cutting or welding shall be permitted only in accordance with the applicable standards for air and gas welding in building construction.

3.812.5 CONCRETE FORMS - Combustible materials shall not be stored on any floor of a building under construction until all combustible concrete forms are removed from the tier immediately above.

3.812.6 FIRE-EXTINGUISHING EQUIPMENT - Required fire extinguishers, water buckets, auxiliary fire-fighting tools or other portable extinguishing equipment shall be installed and maintained on all floors of a construction operation in accessible locations as required in Part 3 Section 7 of this Code (clauses 3.713.1 to 3.713.7) and the Fire Prevention Regulations.

3.812.7 STANDPIPES AND FIRE LINES - Where standpipes are provided as a permanent part of the building, they shall be installed and made ready for instant use of the fire department as the structure progresses in accordance with the provisions of Part 3 Section 7 of this Code. Free access from the street to such standpipes shall be maintained at all times; and materials shall not be stored within one and one-half (1.5 m) metres of any fire hydrant or in the roadway between such hydrant and the centre line of the street.

3.812.8 HOUSEKEEPING - Rubbish and trash shall not be allowed to accumulate on the site and shall be removed as fast as conditions warrant; combustible rubbish shall be removed daily, and shall not be disposed of by burning on the premises or in the immediate vicinity, and the entire premises and area adjoining and around the operation shall be kept in a safe and sanitary condition and free of accumulations of trash, rubbish, nuts, bolts, small tools and other equipment.

3.813 Health Hazards

3.813.1 GENERAL - Every construction or maintenance operation which results in the diffusion of dust, stone and small particles, toxic gases or other harmful substances in quantities hazardous to health shall be safeguarded by means of local ventilation or other protective devices to insure the safety of the public as required by the regulations of the Local Authority and/or the Health Department.

3.813.2 REMOVAL OF DUST - Dust, sand blasts or other harmful agents, when employed or occuring in construction operations, shall be disposed of at or near the point of origin to prevent their diffusion over adjoining premises or streets.

3.813.3 PROTECTIVE EQUIPMENT - Facilities shall be provided for housing the necessary vision, respiratory and protective equipment required in welding or other operations, in approved close containers and in accordance with the regulations of the Local Authority or Health Department.

3.813.4 SANITATION - Every building in the course of demolition, erection, or repair shall be provided with toilet and drinking water facilities which shall be constructed and installed in accordance with the Plumbing Code, Part 4 Section 5 of this Code.

3.814 Welding Safety Precautions

3.814.1 WELDING ENCLOSURES - All welding and flame-cutting operations shall be performed in protected areas with full consideration to safety and fire hazards. Such closed spaces shall be properly ventilated while welding or cutting is being done. Suitable protection against the rays of the electric arc shall be maintained by the contractor where arc-welding operations might be viewed within harmful range by persons other than the welding operators and inspectors.

3.814.2 FLAMMABLE MATERIALS - Proper precautions shall be taken to avoid all risk of fire or explosion, and flammable or explosive materials shall not be stored in the vicinity of welding or cutting operations.

3.814.3 QUALIFICATION OF WELDERS - Gas welding or cutting and arc welding in all building operations shall be carried out by experienced, qualified and "ticketed" welders.

3.814.4 SAFETY METHODS - GENERAL - Before steel beams or other structural shapes or members are cut by means of gas flame (or other means) they shall be secured by chains or cables to prevent parts falling or swinging.

3.814.5 Unless absolutely unavoidable gas or arc welding or cutting shall not be carried out above other workers on the construction site. When unavoidable a non-combustible shield shall be provided between the work and the workers below.

3.814.6 Tanks of fuel gas for welding shall not be moved or allowed to stand for any extended period when not in use without having safety caps. Suitable cradles shall be used for lifting or lowering pressure cylinders or fuel gas containers and hemp rope slings shall not be allowed for this purpose.

3.814.7 Tanks or "bottles" supplying gases for welding or cutting shall be located at no greater distance from the work than necessary, and they shall be securely fastened in place, in an upright position, and not exposed to the direct rays of the sun.

3.815 **Signs 1 - Exemptions from Permit Obligations**

3.815.1 EXEMPTIONS FROM PERMIT OBLIGATIONS - A permit shall not be required for any signs covered by clauses 3.815.2 to 3.815.6 below.

3.815.2 WALL SIGNS - A sign painted on the surface of a fence or approved building wall; or any non-illuminated wall sign on a building or structure which is not more than one square metre (1 m^2) in area.

3.815.3 SALE OR RENT - Signs erected to announce the sale or rent of the property so designated, provided such signs are not more than two square metres (2 m^2) in area.

3.815.4 TRANSIT DIRECTIONS - The erection or maintenance of a sign designating the location of a transit line, a railroad station or other public carrier when not more than one-half of one square metre in area (0.5 m^2).

3.815.5 STREET SIGNS - Signs erected by a jurisdiction for street direction.

3.815.6 PROJECTING SIGNS - A projecting sign not exceeding one quarter of one square metre (0.25 m^2) of display surface.

3.815.7 EGRESS OBSTRUCTIONS - A sign shall not be erected, constructed or maintained so as to obstruct any fire escape, required exitway, window or door opening used as an element of a means of egress, nor shall it prevent free passage from one part of a roof to another part or access thereto as required by the provisions of Part 3 Section 5 of this Code.

3.815.8 OBSTRUCTION TO VENTILATION - A sign shall not be attached in any form, shape or manner which will interfere with any opening required for ventilation by Part 3 Section 4, except that such signs may be erected in front of and may cover transom windows when not in violation of the provisions of this code.

3.815.9 PROJECTING SIGNS - A projecting sign erected at other than right angles to the walls of a building or structure outside of the building line which extends above the roof cornice or parapet wall and which obstructs access to the roof is hereby deemed unlawful. Such signs shall be reconstructed or removed as herein required.

3.815.10 REMOVING OR RECONSTRUCTING SIGNS - A sign heretofore approved and erected shall not be repaired, altered or moved, nor shall any sign, or any substantial part thereof, which is blown down, destroyed or removed be re-erected, reconstructed, rebuilt or relocated unless it is made to comply with all applicable requirements of this section of the Code.

3.815.11 REPAIR OF UNSAFE SIGNS - This section shall not be construed to prevent the repair or restoration to a safe condition as directed by the Local Authority of any part of an existing sign when damaged by storm or other accidental emergency.

3.815.12 RELOCATING SIGNS - Any sign that is moved to another location either on the same or to other premises shall be considered a new sign and a permit shall be secured for any work performed in connection therewith.

3.815.13 MAINTENANCE - All signs for which permit is required, together with all their supports, braces, guys, and anchors shall be kept in repair in accordance with the provisions of this article and when not galvanized or constructed of approved corrosion-resistive non-combustible materials shall be painted when necessary to prevent corrosion.

3.815.14 It shall be the duty and responsibility of the owner or lessee of every sign to maintain the ground and/or immediate premises occupied by the sign in a tidy, clean, sanitary, and healthful condition.

3.815.15 Every sign shall be subject to the inspection and approval, from time to time by inspectors of the Local Controlling Authority.

3.815.16 CONSTRUCTION - ALL SIGNS - All signs shall be designed and constructed in conformity to the provisions for materials, loads and stresses covered in Part 2 of this code (Structural and Foundation Loads and Stresses, and also Materials).

3.815.17 DESIGN LOADS - All signs shall be designed and constructed to withstand wind pressures as provided in Part 2 Section 2 and the Table of Effective Velocity (wind) Pressures in that section.

3.815.18 EARTHQUAKE LOADS - Signs adequately designed to withstand wind pressures as above, shall generally be considered capable of withstanding earthquake shocks.

3.815.19 ILLUMINATION - A sign shall not be illuminated by other than electrical means. All electrical devices and wiring shall be installed in accordance with the requirements of the relevant local standard for electrical wiring and equipment. No open spark or flame shall be used for display purposes unless specifically approved by the Building Authority for locations outside of the fire limits.

3.815.20 USE OF COMBUSTIBLES - The following clauses 3.815.21 and 3.815.22 shall apply to combustible materials for signs.

3.815.21 ORNAMENTAL FEATURES - Wood or approved plastic or other materials of combustible characteristics similar to wood may be used for moldings, cappings, nailing blocks, letters and latticing when permitted in sub-section 3.817 below, and for other purely ornamental features of signs in accordance with the approved rules.

3.815.22 SIGN FACING - Sign facing may be made of approved combustible plastic providing the area of such facing section is not more than 10 square metres (10 m^2) and the wiring for electric lighting is entirely enclosed in the sign cabinet with a clearance of not less than 50 millimetres from the facing material.

3.815.23 SERVICING DEVICES - Ladders, platforms, hooks, rings and all other devices for the use of servicing personnel shall have safety devices and design loading in accordance with safety requirements of the Local Inspectorate for Safety.

3.815.24 ANIMATED DEVICES - Signs which contain moving sections or ornaments shall have fail-safe provisions to prevent the section or ornament from releasing and falling or shifting its centre of gravity more than 350 millimetres. The fail-safe device shall be in addition to the mechanism and its housing which operate the movable section or ornament. The fail-safe device shall be capable of supporting the full dead weight of the section or ornament when the moving mechanism releases.

3.816 Signs 2: Liability, Insurance and Bonds

3.816.1 FILING - The Local Authority, before issuing a permit for any sign to be erected will assess (taking into account the size, height locality, and position of proposed sign) whether there may be any danger to the public if such sign were to fall, be blown down, or be dislodged from its position by any other accidental means. If so required by the Local Authority after due consideration of above possibilities, the owner/applicant shall file for a suitable life and/or accident insurance policy, or he shall pay over a bond in the amount agreed between the Local Authority and Insurance Company.

3.816.2 CONDITIONS - Such bond, or insurance policy shall protect the Local Authority/jurisdiction from any and all claims, or demands for damages, by reason of any negligence of the sign maker/hanger or his agents, or by reason of defects in construction, or damages resulting from collapse, failure or combustion of the sign or any parts thereof.

3.816.3 NOTICE OF CANCELLATION - The obligation herein specified shall remain in force and effect during the life of every sign and shall not be cancelled by the principal or surety until after thirty (30) days notice to the Building Authority and then only on written assurance that the sign has been demolished or removed.

3.817 Signs 3: Illuminated, Projecting, Wall and Roof Mounted

3.817.1 ILLUMINATED SIGNS: CERTIFICATES - All electrically illuminated signs shall be certified as to electric wiring and devices by the authoritative agency having jurisdiction, and all wiring and accessory electrical equipment shall conform to the requirements of the relevant local standards.

3.817.2 ADDITIONAL PERMITS - Electrical permits shall be issued for the erection or maintenance of illuminated signs, as well as a permit from the Local Authority.

3.817.3 RE-LETTERING SIGNS - The requirements of this section shall not apply to the re-lettering of illuminating signs, except where such re-lettering requires a change of wiring of the sign.

3.817.4 PROJECTING SIGNS: MATERIALS - Projecting signs shall be constructed entirely of metal or other approved non-combustible materials except as provided in the general requirements for all signs, sub-section 3.815 above.

3.817.5 MAXIMUM PROJECTION - A projecting sign shall not extend beyond a vertical plane one half of one metre (0.5 m) inside the line of curb.

3.817.6 CLEARANCES - A clear space of not less than three metres (3.0 m) shall be provided below all such parts of electrical signs unless a special dispensation has been given by the Local Authority.

3.817.7 WALL SIGNS - MATERIALS - Wall signs which have an area exceeding ten square metres (10.0 m^2) shall be constructed of metal or other approved non-combustible materials, except for nailing rails and as provided in sub-section 3.815 above.

3.817.8 EXTENSION - Wall signs shall not be erected to extend above the top of the wall, nor extend byond the ends of the wall to which they are attached, unless meeting all the requirements for roof signs, projecting signs or ground signs as the case may be.

3.817.9 ROOF SIGNS - MATERIALS - All roof signs shall be constructed entirely of metal or other approved non-combustible materials except as provided in sub-section 3.815. Provisions shall be made for electric grounding of all metallic parts; and where combustible materials are permitted in letters or other ornamental features, all wiring and conduit shall be kept free and insulated therefrom.

3.817.10 BOTTOM CLEARANCE - There shall be a clear space of not less than one and one-half metres (1.5 m) between the lowest part of the sign and the roof level, except for necessary structural supports.

3.817.11 CLOSED SIGNS - A closed roof sign shall not be erected to a height greater than fifteen metres (15.0 m) above the roof of Types 1 and 2 constructed buildings nor more than ten metres (10 m) of buildings of Type 3 and 4 construction.

3.817.12 OPEN SIGNS - An open roof sign shall not exceed a height of thirty metres (30.0 m) above the roof of buildings of Types 1 and 2 construction; and not more than 20 metres above the roof of buildings of Type 3 and 4 construction.

3.817.13 COMBUSTIBLE SUPPORTS - Within the fire limits, a roof sign which exceeds twelve metres (12.0 m) in height shall not be supported on or braced to wooden beams or other combustible construction of a building or structure unless otherwise approved by the Building Authority.

3.817.14 GROUND SIGNS (REFER ALSO 3.815.21 ABOVE) BOTTOM CLEARANCE - The bottom capping of all ground signs shall be at least three-quarters of one metre (0.75 m) = (750 mm) above the ground but the intervening space may be filled with open lattice work or platform decorative trim.

3.817.15 FIRE LIMITS - In the fire limits, a ground sign shall not be constructed of combustible materials, except as provided in clause 3.815.21 above.

3.817.16 OUTSIDE FIRE LIMITS - Outside the fire limits, the structural frame of ground signs shall not be erected of combustible materials to a height of more than ten metres (10 m) above the ground.

3.817.17 MAXIMUM SIZE - In all locations, when constructed entirely of non-combustible material, ground signs may be erected to a height of thirty metres (30.0 m) above the ground; and to greater heights when approved by the building official and located so as not to create hazard or danger to the public.

3.817.18 ADDITIONAL LOADS - Projection sign structures or ground or roof signs which could be used to support an individual on a ladder or other servicing device whether or not specifically designed for the servicing device shall be capable of supporting the anticipated additional load but in no case less than fifty kilograms (50 kg) concentrated horizontal load and one hundred and forty kilograms (140 kg) vertical concentrated load applied at the point of assumed loading or point of most eccentric loading. The building component to which the projecting sign is attached shall also be designed to support the additional loads.

3.817.19 PORTABLE SIGNS - Portable signs shall conform to all requirements for ground, roof, projecting flat and temporary signs when they are used in a similar capacity. The stipulations in this section shall not be construed as to require portable signs to have connections to surfaces, tie-downs or foundations when provisions are made by temporary means or configuration of the structure or the ground to provide stability for the expected duration of the installation.

3.817.20 ELECTRICAL - Portable signs which require electrical service shall have a positive connecting device on the sign. Electrical service lines to the sign shall be protected from damage from all anticipated traffic.

3.817.21 MISCELLANEOUS AND TEMPORARY SIGNS - Temporary signs and banners attached to or suspended from a building or poles, trees, or other anchor points and constructed of cloth or other combustible material shall be strongly constructed and shall be securely attached to their supports. They shall be removed as soon as torn or damaged, and not later than sixty (60) days after erection except that permits for temporary signs suspended from or attached to a canopy or marquee shall be limited to a period of ten days.

3.817.22 MAXIMUM SIZE - Temporary signs of combustible construction shall be not more than three metres (3.0 m) in one (1) dimension nor more than forty-five square metres (45 m^2) in area.

3.817.23 SUPPORTS - When more than ten square metres (10 m^2) in area, temporary signs and banners shall be constructed and fastened to supports capable of withstanding the design loads listed in Part 2 Section 1 of this Code.

3.817.24 SPECIAL PERMITS - Temporary signs used for holiday, public demonstrations or promotion of civil welfare or charitable purposes which extend across streets or other public spaces, shall be subject to special approval of the authority having jurisdiction.

FACTORS FOR CONVERTING IMPERIAL UNITS TO SI UNITS

LENGTH

Feet (ft) x 0.3048*	= Meters (m)
Inches (in) x 0.0254*	= Meters (m)
Miles (mi) x 1609.34	= Meters (m)
Miles (mi) x 1.60934	= Kilometers (km)
Nautical Miles x 1852.0*	= Meters (m)
Yards (yd) x 0.9144*	= Meters (m)

AREA

$Feet^2$ (ft^2) x 0.0929030	= $Meters^2$ (m^2)
Acres x 4046.86	= $Meters^2$ (m^2)
$Miles^2$ (mi^2) x 2.58999	= $Kilometers^2$ (km^2)

VOLUME

$Feet^3$ (ft^3) x 0.02831685	= $Meters^3$ (m^3)
$Feet^3$ (ft^3) x 28.31685	= Liters**
$Yards^3$ (yd^3) x 0.764555	= $Meters^3$ (m^3)
Acre feet x 1233.48	= $Meters^3$ (m^3)
Gallons (gal) x 0.00454609	= $Meters^3$ (m^3)
Gallons (gal) x 4.54609	= Liters**

VELOCITY

Feet per second (ft/s) x 0.3048*	= Meters per second (m/s)
Miles per hour (mi/hr) x 0.44704*	= Meters per second (m/s)
Miles per hour (mi/hr) x 1.60934	= Kilometers per hour (km/hr)
Knots x 0.514444	= Meters per second (m/s)
Knots x 1.852*	= Kilometers per hour (km/hr)

DISCHARGE

$Feet^3$ per second (ft^3/s) x 0.02831685	= $Meters^3$ per second (m^3/s)
Millions of gallons per day (mgd) x 0.0526167	= $Meters^3$ per second (m^3/s)
Acre-feet per day x 0.0142764	= $Meters^3$ per second (m^3/s)
Gallons per minute (gal/min) x 0.0000757680	= $Meters^3$ per second (m^3/s)

FORCE

Pounds (lb) x 0.453592	= Kilogyrams force (kgf)
Pounds (lb) x 453.592	= Grams (g)
Pounds (lb) x 4.44822	= Newtons** (N)
Tons x 0.907185	= Metric tons**

PRESSURE

Pounds per foot^2 (lb/ft^2) x 47.8803	= Newtons per meter^2 (N/m^2)
Pounds per foot^2 (lb/ft^2) x 4.88243	= Kilograms force per meter^2 (kgf/m^2)
Pounds per inch^2 (lb/in^2) x 6894.76	= Newtons per meter^2 (N/m^2)
Pounds per inch^2 (lb/in^2) 0.00689476 x 10	= Newtons per millimeter^2 (N/mm^2)
	= Megapascal (MPa)
Pounds per inch^2 (lb/in^2) x 703.070	= Killograms force per meter^2 (kgf/m^2)
Millibars (mb) x 100.0*	= Newtons per meter^2 (N/m^2)

UNIT WEIGHT

Pounds per foot^3 (lb/ft^3) x 157.0876	= Newtons per meter^3 (N/m^3)
Pounds per foot^3 (lb/ft^3) x 16.0185	= Kilograms force per meter^3 (kgf/m^3)
Pounds per foot^3 (lb/ft^3) x 0.0160185	= Grams per centimeter^3 (g/cm^3)

MASS AND DENSITY

Slugs x 14.5939	= Kilograms (kg)
Slugs per foot^3 x 515.379	= Kilograms per meter^3 (kg/m^3)

VISCOSITY

Pound-seconds per foot^2 ($lb\text{-}s/ft^2$) or slugs per foot second x 47.8803	= Newtons seconds per meter^2 (Ns/m^2)**
Feet^2 per second (ft^2/s) x 0.092903	= Meters^2 per second (m^2/s)

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* Exact values

** Liters x 1000.0 = cemtimeters^3
Liters x 0.001 = meters^3
Metric tons x 1000.0 = kilograms force
Kilograms force x 9.80665 = newtons
Newtons x 100,000.0 = dynes
Newton seconds per meter^2 x 0.1 = poises

SI PREFIXES AND MEANINGS

Prefix (abbreviation)	Meaning
Mega- (M)	1,000,000.
Kilo- (k)	1,000.
Hecto- (h)	100.
Deka- (da)	10.
Deci- (d)	0.1
Centi- (c)	0.01
Milli- (m)	0.001
Micro- (μ)	0.000001

NOTES