

DEFINITIONS AND GENERAL REQUIREMENTS

1.1 INTRODUCTION

1.1.1 SCOPE

The definitions providing meanings of different terms and general requirements for the structural design of buildings, structures, and components thereof are specified in this chapter. These requirements shall apply to all buildings and structures or their components regulated by this code. All anticipated loads required for structural design shall be determined in accordance with the provisions of Chapter 2. Design parameters required for the structural design of foundation elements shall conform to the provisions of Chapter 3. Design of structural members using various construction materials shall comply with the relevant provisions of Chapters 4 through 13. The FPS equivalents of the empirical expressions used throughout Part 6 are listed in Appendix A.

This Code shall govern in all matters pertaining to design, construction, and material properties wherever this Code is in conflict with requirements contained in other standards referenced in this Code. However, in special cases where the design of a structure or its components cannot be covered by the provisions of this code, other relevant internationally accepted codes referred in this code may be used.

1.1.2 DEFINITIONS

The following definitions shall provide the meaning of certain terms used in this chapter.

BASE SHEAR : Total design lateral force or shear at the base of a structure.

BASIC WIND SPEED : Three-second gust speed at 10 metres above the mean ground level in terrain Exposure-B defined in Sec 2.4.8 and associated with an annual probability of occurrence of 0.02.

BEARING WALL SYSTEM : A structural system without a complete vertical load carrying space frame.

BRACED FRAME : An essentially vertical truss system of the concentric or eccentric type which is provided to resist lateral forces.

BUILDING FRAME SYSTEM : An essentially complete space frame which provides support for loads.

CONCENTRIC BRACED FRAME (CBF) : A steel braced frame designed in conformance with Sec 10.20.13. or 10.20.14.

COLLECTOR : A member or element used to transfer lateral forces from a portion of a structure to the vertical elements of the lateral force resisting elements.

BUILDINGS : Structures that enclose a space and are used for various occupancies.

DEAD LOAD : The load due to the weight of all permanent structural and nonstructural components of a building or a structure, such as walls, floors, roofs and fixed service equipment.

DIAPHRAGM : A horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes horizontal bracing systems.

DUAL SYSTEM : A combination of Moment Resisting Frames and Shear Walls or Braced Frames to resist lateral loads designed in accordance with the criteria of Sec 1.3.2.

ECCENTRIC BRACED FRAME (EBF) : A steel braced frame designed in conformance with Sec 10.20.15.

HORIZONTAL BRACING SYSTEM : A horizontal truss system that serves the same function as a floor or roof diaphragm.

INTERMEDIATE MOMENT RESISTING FRAME (IMRF) : A concrete moment resisting frame designed in accordance with Sec 8.3.10.

LIVE LOAD : The load superimposed by the use and occupancy of a building.

MOMENT RESISTING FRAME : A frame in which members and joints are capable of resisting forces primarily by flexure.

ORDINARY MOMENT RESISTING FRAME (OMRF) : A moment resisting frame not meeting special detailing requirements for ductile behaviour.

PRIMARY FRAMING SYSTEM : That part of the structural system assigned to resist lateral forces.

SHEAR WALL : A wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as a vertical diaphragm or a structural wall).

SLENDER BUILDINGS AND STRUCTURES : Buildings and structures having a height exceeding five times the least horizontal dimension, or having a fundamental natural frequency less than 1 Hz. For those cases where the horizontal dimensions vary with height, the least horizontal dimension at mid height shall be used.

SOFT STOREY : Storey in which the lateral stiffness is less than 70 per cent of the stiffness of the storey above.

SPACE FRAME : A three-dimensional structural system without bearing walls composed of members interconnected so as to function as a complete self contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

SPECIAL MOMENT RESISTING FRAME (SMRF) : A moment resisting frame specially detailed to provide ductile behaviour complying with the requirements of Chapter 8 or 10 for concrete or steel frames respectively.

SPECIAL STRUCTURAL SYSTEM : A structural system not listed in Table 1.3.1 and specially designed to carry the lateral loads. See Sec 1.3.2.5.

STOREY : The space between any two floor levels including the roof of a building. Storey-x is the storey below level x.

STOREY SHEAR, V_x : The summation of design lateral forces above the storey under consideration.

STRENGTH : The usable capacity of an element or a member to resist the load as prescribed in these provisions.

TERRAIN : The ground surface roughness condition when considering the size and arrangement of obstructions to the wind.

THREE-SECOND GUST SPEED : The highest average wind speed over a 3 second duration at a height of 10 m. The three-second gust speed is derived using Durst's model in terms of the mean wind speed and turbulence intensity.

TOWER : A tall, slim vertical structure.

VERTICAL LOAD-CARRYING FRAME : A space frame designed to carry all vertical gravity loads.

WEAK STOREY : Storey in which the lateral strength is less than 80 per cent of that of the storey above.

1.1.3 SYMBOLS AND NOTATION

The following symbols and notation shall apply to the provisions of this chapter:

D	=dead load on a member including self weight and weight of components, materials and permanent equipments supported by the member
E	=earthquake load
F_i	=lateral force applied at level- i of a building
h	=height of a building or a structure above ground level in metres
h_i, h_n, h_x	=height in metres above ground level to level- i , - n or - x respectively
level- i	= i -th level of a structure above the base; $i=1$ designates the first level above the base
level- n	=upper most level of a structure
level- x	= x -th level of a structure above the base; $x=1$ designates the first level above the base.
L	=live load due to intended use or occupancy
l	=span of a member or component.
M_x	=overturning moment at level- x
V	=the total design lateral force or shear at the base
V_x	=the storey shear at storey level- x
R	=response modification or reduction coefficient for structural system given in Table 2.5.7 for seismic design.
S	= Snow load
T	= Fundamental period of vibration in seconds
W	= Load due to wind pressure.
W'	= Weight of an element or component
Z	= Seismic zone coefficient given in Fig 2.5.1 or Tables 2.5.2 or 2.5.3
Δ	= Storey lateral drift.

1.2 BASIC CONSIDERATIONS

1.2.1 GENERAL

All buildings and structures shall be designed and constructed in conformance with the provisions of this section. The buildings and portions thereof shall support all loads including dead load specified in this chapter and elsewhere in this Code. Impact, fatigue and self-straining forces shall be considered where these forces occur.

1.2.2 BUILDINGS AND STRUCTURES

A structure shall ordinarily be described as an assemblage of framing members and components arranged to support both gravity and lateral forces. Structures may be classified as building and non-building structures. Structures that enclose a space and are used for various occupancies shall be called buildings or building structures. Structures other than buildings, such as water tanks, bridges, communication towers, chimneys etc., shall be called

non-building structures. When used in conjunction with the word building(s), the word structure(s) shall mean non-building structures, e.g. 'buildings and structures' or 'buildings or structures'. Otherwise the word 'structures' shall include both buildings and non-building structures.

1.2.3 BUILDING AND STRUCTURE OCCUPANCY CATEGORIES

Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 1.2.1 for the purposes of applying flood, surge, wind and earthquake provisions. The occupancy categories range from I to IV, where Occupancy Category I represents buildings and other structures with a low hazard to human life in the event of failure and Occupancy Category IV represents essential facilities. Each building or other structure shall be assigned to the highest applicable occupancy category or categories. Assignment of the same structure to multiple occupancy categories based on use and the type of load condition being evaluated (e.g., wind or seismic) shall be permissible.

When buildings or other structures have multiple uses (occupancies), the relationship between the uses of various parts of the building or other structure and the independence of the structural systems for those various parts shall be examined. The classification for each independent structural system of a multiple-use building or other structure shall be that of the highest usage group in any part of the building or other structure that is dependent on that basic structural system.

1.2.4 SAFETY

Buildings, structures and components thereof, shall be designed and constructed to support all loads, including dead loads, without exceeding the allowable stresses or specified strengths (under applicable factored loads) for the materials of construction in the structural members and connections.

1.2.5 SERVICEABILITY

Structural framing systems and components shall be designed with adequate stiffness to have deflections, vibration, or any other deformations within the serviceability limit of building or structure. The deflections of structural members shall not exceed the more restrictive of the limitations provided in Chapters 2 through 13 or that permitted by Table 1.2.2 or the notes that follow. For wind and earthquake loading, story drift and sway shall be limited in accordance with the provisions of Sec 1.5.6.

1.2.6 RATIONALITY

Structural systems and components thereof shall be analyzed, designed and constructed based on rational methods which shall include, but not be limited to, the provisions of Sec 1.2.7

1.2.7 ANALYSIS

Analysis of the structural systems shall be made for determining the load effects on the resisting elements and connections, based on well established principles of mechanics taking equilibrium, geometric compatibility and both short and long term properties of the construction materials into account and incorporating the following:

1.2.7.1 MATHEMATICAL MODEL

A mathematical model of the physical structure shall represent the spatial distribution of stiffness and other properties of the structure which is adequate to provide a complete load path capable of transferring all loads and forces from their points of origin to the load-resisting elements for obtaining various load effects. For dynamic analysis, mathematical model shall also incorporate the appropriately distributed mass and damping properties of

the structure adequate for the determination of the significant features of its dynamic response. All buildings and structures shall be thus analyzed preferably using a three dimensional computerized model incorporating these features of mathematical model. It is essential to use three dimensional computer model to represent a structure having irregular plan configuration such as those listed in Tables 1.3.2 and 1.3.3 and having rigid or semirigid floor and roof diaphragms. Requirements for two-dimensional model and three dimensional models for earthquake analysis are described in Sec.2.5.11 to 2.5.14.

Table 1.2.1 Occupancy Category of Buildings and Other Structures for Flood, Surge, Wind and Earthquake Loads

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with daycare facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations^a • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency • Aviation control towers, air traffic control centers, and emergency aircraft hangars • Water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.	IV

^aCogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.

1.2.7.2 LOADS AND FORCES

All prescribed loads and forces to be supported by the structural systems shall be determined in accordance with the applicable provisions of this chapter and Chapter 2. Loads shall be applied on the mathematical model specified in Sec. 1.2.7.1 at appropriate spatial locations and along desired directions.

Table 1.2.2 Deflection Limits (Except earthquake load)

CONSTRUCTION	L	W^f	$D + L^{d, g}$
Roof members: ^e			
Supporting plaster ceiling	$l/360$	$l/360$	$l/240$
Supporting nonplaster ceiling	$l/240$	$l/240$	$l/180$
Not supporting ceiling	$l/180$	$l/180$	$l/120$
Floor members	$l/360$	—	$l/240$
Exterior walls and interior partitions:			
With brittle finishes	—	$l/240$	—
With flexible finishes	—	$l/120$	—
Farm buildings	—	—	$l/180$
Greenhouses	—	—	$l/120$

In the above table l stands for span of the member under consideration; L stands for live load, W stands for wind load and D stands for dead load.

Notes:

- a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed $l/60$. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed $l/150$. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed $l/90$. For roofs, this exception only applies when the metal sheets have no roof covering.
- b. Interior partitions not exceeding 2m in height and flexible, folding and portable partitions are not governed by the provisions of this section.
- c. For cantilever members, l shall be taken as twice the length of the cantilever.
- d. For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from $L + 0.5D$ is permitted to be substituted for the deflection resulting from $L + D$.
- e. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Section 1.6.5 for rain and ponding requirements.
- f. The wind load is permitted to be taken as 0.7 times the “component and cladding” loads for the purpose of determining deflection limits herein.
- g. For steel structural members, the dead load shall be taken as zero.
- h. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed $l/60$. For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed $l/175$ for each glass lite or $l/60$ for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed $l/120$.

1.2.7.3 SOIL-STRUCTURE INTERACTION

Soil-structure interaction effects, where required, shall be included in the analysis by appropriately including the properly substantiated properties of soil into the mathematical model specified in Sec. 1.2.7.1 above.

1.2.8 DISTRIBUTION OF HORIZONTAL SHEAR

The total lateral force 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 systems or diaphragms.

1.2.9 HORIZONTAL TORSIONAL MOMENTS

Structural systems and components shall be designed to sustain additional forces resulting from torsion due to eccentricity between the centre of application of the lateral forces and the centre of rigidity of the lateral force resisting system. Forces shall not be decreased due to torsional effects. For accidental torsion effects on seismic forces, requirements shall conform to Sec 2.5.9.6

1.2.10 STABILITY AGAINST OVERTURNING AND SLIDING

Every building or structure shall be designed to resist the overturning and sliding effects caused by the lateral forces specified in this chapter.

1.2.11 ANCHORAGE

Anchorage of the roof to wall and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces resulting from the application of the prescribed loads. Additional requirements for masonry or concrete walls shall be those given in Sec 1.7.3.6.

1.2.12 GENERAL STRUCTURAL INTEGRITY

Buildings and structural systems shall possess general structural integrity, that is the ability to sustain local damage caused due to misuse or accidental overloading, with the structure as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage.

1.2.13 PROPORTIONING OF STRUCTURAL ELEMENTS

Structural elements, components and connections shall be proportioned and detailed based on the design methods provided in the subsequent chapters for various materials of construction, such as reinforced concrete, masonry, steel etc. to resist various load effects obtained from a rational analysis of the structural system.

1.2.14 WALLS AND FRAMING

Walls and structural framing shall be erected true and plumb in accordance with the design. Interior walls, permanent partitions and temporary partitions exceeding 1.8 m of height shall be designed to resist all loads to which they are subjected. If not otherwise specified elsewhere in this Code, walls shall be designed for a minimum load of 0.25 kN/m² applied perpendicular to the wall surfaces. The deflection of such walls under a load of 0.25 kN/m² shall not exceed $\frac{1}{240}$ of the span for walls with brittle finishes and $\frac{1}{120}$ of the span for walls with flexible finishes. However, flexible, folding or portable partitions shall not be required to meet the above load and deflection criteria, but shall be anchored to the supporting structure.

1.2.15 ADDITIONS TO EXISTING STRUCTURES

When an existing building or structure is extended or otherwise altered, all portions thereof affected by such cause shall be strengthened, if necessary, to comply with the safety and serviceability requirements provided in Sec 1.2.4 and 1.2.5 respectively.

1.2.16 PHASED CONSTRUCTION

When a building or structure is planned or anticipated to undergo phased construction, structural members therein shall be investigated and designed for any additional stresses arising due to such construction.

1.2.17 LOAD COMBINATIONS AND STRESS INCREASE

Every building, structure, foundation or components thereof shall be designed to sustain, within the allowable stress or specified strength (under factored load), the most unfavourable effects resulting from various combinations of loads specified in section 2.7. Except otherwise permitted or restricted by any other section of this Code, maximum increase in the allowable stress shall be 33% when allowable or working stress method of design is followed. For soil stresses due to foundation loads, load combinations and stress increase specified in Sec 2.7.4 for allowable stress design method shall be used.

1.3 STRUCTURAL SYSTEMS

1.3.1 GENERAL

Every structure shall have one of the basic structural systems specified in Sec 1.3.2 or a combination thereof. The structural configuration shall be as specified in Sec 1.3.4 with the limitations imposed in Sec 2.5.7.4.

1.3.2 BASIC STRUCTURAL SYSTEMS

Structural systems for buildings and other structures shall be designated as one of the types A to G listed in Table 1.3.1. Each type is again classified as shown in the table by the types of vertical elements used to resist lateral forces. A brief description of different structural systems are presented in following sub-sections.

Table 1.3.1: Basic Structural Systems

A. BEARING WALL SYSTEMS (no frame) 1. Special reinforced concrete shear walls 2. Ordinary reinforced concrete shear walls 3. Ordinary reinforced masonry shear walls 4. Ordinary plain masonry shear walls
B. BUILDING FRAME SYSTEMS (with bracing or shear wall) 1. Steel eccentrically braced frames, moment resisting connections at columns away from links 2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links 3. Special steel concentrically braced frames 4. Ordinary steel concentrically braced frames 5. Special reinforced concrete shear walls 6. Ordinary reinforced concrete shear walls 7. Ordinary reinforced masonry shear walls 8. Ordinary plain masonry shear walls
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall) 1. Special steel moment frames 2. Intermediate steel moment frames 3. Ordinary steel moment frames 4. Special reinforced concrete moment frames 5. Intermediate reinforced concrete moment frames 6. Ordinary reinforced concrete moment frames
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall) 1. Steel eccentrically braced frames 2. Special steel concentrically braced frames 3. Special reinforced concrete shear walls 4. Ordinary reinforced concrete shear walls
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall) 1. Special steel concentrically braced frames 2. Special reinforced concrete shear walls 3. Ordinary reinforced masonry shear walls 4. Ordinary reinforced concrete shear walls
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE

1.3.2.1 BEARING WALL SYSTEM

A structural system having bearing walls or bracing systems without a complete vertical load carrying frame to support gravity loads. Resistance to lateral loads is provided by shear walls or braced frames.

1.3.2.2 BUILDING FRAME SYSTEM

A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral loads is provided by shear walls or braced frames separately.

1.3.2.3 MOMENT RESISTING FRAME SYSTEM

A structural system with an essentially complete space frame providing support for gravity loads. Moment resisting frames also provide resistance to lateral load primarily by flexural action of members, and may be classified as one of the following types:

- a) Special Moment Resisting Frames (SMRF)
- b) Intermediate Moment Resisting Frames (IMRF)
- c) Ordinary Moment Resisting Frames (OMRF).

The framing system, IMRF and SMRF shall have special detailing to provide ductile behaviour conforming to the provisions of Sec 8.3 and 10.20 for concrete and steel structures respectively. OMRF need not conform to these special ductility requirements of Chapter 8 or 10.

1.3.2.4 DUAL SYSTEM

A structural system having a combination of the following framing systems :

- a) Moment resisting frames (SMRF, IMRF or steel OMRF), and
- b) Shear walls or braced frames.

The two systems specified in (a) and (b) above shall be designed to resist the total lateral force in proportion to their relative rigidities considering the interaction of the dual system at all levels. However, the moment resisting frames shall be capable of resisting at least 25% of the applicable total seismic lateral force, even when wind or any other lateral force governs the design.

1.3.2.5 SPECIAL STRUCTURAL SYSTEM :

A structural system not defined above nor listed in Table 1.3.1 and specially designed to carry the lateral loads, such as tube-in-tube, bundled tube, etc.

1.3.2.6 NON-BUILDING STRUCTURAL SYSTEM

A structural system used for purposes other than in buildings and conforming to Sec 1.5.4.8, 1.5.4.9, 2.4 and 2.5.

1.3.3 COMBINATION OF STRUCTURAL SYSTEMS

When different structural systems of Sec 1.3.2 are combined for incorporation into the same structure, design of the combined seismic force resisting system shall conform to the provisions of Sec 2.5.7.5.

1.3.4 STRUCTURAL CONFIGURATIONS

Based on the structural configuration, each structure shall be designated as a regular or irregular structure as defined below:

1.3.4.1 REGULAR STRUCTURES

Regular structures have no significant physical discontinuities or irregularities in plan or vertical configuration or in their lateral force resisting systems. Typical features causing irregularity are described in Sec 1.3.4.2.

1.3.4.2 IRREGULAR STRUCTURES

Irregular structures have either vertical irregularity or plan irregularity or both in their structural configurations or lateral force resisting systems.

1.3.4.2.1 Vertical Irregularity

Structures having one or more of the irregular features listed in Table 1.3.2 shall be designated as having a vertical irregularity.

Table 1.3.2: Vertical Irregularities of Structures

Vertical Irregularity		Reference*
Type	Definition	Section
I	Stiffness Irregularity (Soft Storey) : A soft storey is one in which the lateral stiffness is less than 70 per cent of that in the storey above or less than 80 per cent of the average stiffness of the three storeys above.	2.5.7 to 2.5.14 and 2.5.19
II	Mass Irregularity : Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150 per cent of the effective mass of an adjacent storey. A roof which is lighter than the floor below need not be considered.	2.5.7 to 2.5.14
III	Vertical Geometric Irregularity : Vertical geometric irregularity shall be considered to exist where horizontal dimension of the lateral force-resisting system in any storey is more than 130 per cent of that in an adjacent storey, one-storey penthouses need not be considered.	2.5.7 to 2.5.14
IV	In-Plane Discontinuity in Vertical Lateral Force-Resisting Element : An in-plane offset of the lateral load-resisting elements greater than the length of those elements.	2.5.7 to 2.5.14
Va	Discontinuity in Capacity (Weak Storey) : A weak storey is one in which the storey strength is less than 80 per cent of that in the storey above. The storey strength is the total strength of all seismic-resisting elements sharing the storey shear for the direction under consideration.	2.5.7 to 2.5.14 and 2.5.19
Vb	Extreme Discontinuity in Capacity (Very Weak Storey) : A very weak storey is one in which the storey strength is less than 65 per cent of that in the storey above.	2.5.7 to 2.5.14 and 2.5.19

1.3.4.2.2 Plan Irregularity

Structures having one or more of the irregular features listed in Table 1.3.3 shall be designated as having a plan irregularity.

Table 1.3.3: Plan Irregularities of Structures

Plan Irregularity		Reference*
Type	Definition	Section
I	Torsional Irregularity (to be considered when diaphragms are not flexible) : Torsional irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion, at one end of the structure is more than 1.2 times the average of the storey drifts at the two ends of the structure.	2.5.7 to 2.5.14

II	Reentrant Corners : Plan configurations of a structure and its lateral force-resisting system contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 15 per cent of the plan dimension of the structure in the given direction.	2.5.7 to 2.5.14
III	Diaphragm Discontinuity : Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 per cent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 per cent from one storey to the next.	2.5.7 to 2.5.14
IV	Out-of-plane Offsets : Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	2.5.7 to 2.5.14
V	Nonparallel Systems : The vertical lateral load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	2.5.7 to 2.5.14

1.4 DESIGN FOR GRAVITY LOADS

1.4.1 GENERAL

Design of buildings and components thereof for gravity loads shall conform to the requirements of this section. Gravity loads, such as dead load and live loads applied at the floors or roof of a building shall be determined in accordance with the provisions of Chapter 2.

1.4.2 FLOOR DESIGN

Floor slabs and decks shall be designed for the full dead and live loads as specified in Sec 2.2 and 2.3 respectively. Floor supporting elements such as beams, joists, columns etc. shall be designed for the full dead load and the appropriately reduced live loads set forth by the provisions of Sec 2.3.13. Design of floor elements shall also conform to the following provisions :

- a) **Uniformly Distributed Loads :** Where uniform floor loads are involved, consideration may be limited to full dead load on all spans in combination with full live load on adjacent spans and on alternate spans to determine the most unfavourable effect of stresses in the member concerned.
- b) **Concentrated Loads :** Provision shall be made in designing floors for a concentrated load as set forth in Sec 2.3.5 applied at a location wherever this load acting upon an otherwise unloaded floor would produce stresses greater than those caused by the uniform load required therefore.
- c) **Partition Loads :** Loads due to permanent partitions shall be treated as a dead load applied over the floor as a uniform line load having an intensity equal to the weight per metre run of the partitions as specified in Sec 2.2.5. Loads for light movable partitions shall be determined in accordance with the provisions of Sec 2.3.6.
- d) **Design of Members :** Floor members, such as slabs or decks, beams, joists etc. shall be designed to sustain the worst effect of the dead plus live loads or any other load combinations as specified in Sec 2.7. Where floors are used as diaphragms to transmit lateral loads between various resisting elements, those loads shall be determined following the provisions of Sec 1.7.3.8. Detailed design of the floor elements shall be performed using the procedures provided in Chapters 4 through 13 for various construction materials.

1.4.3 ROOF DESIGN

Roofs and their supporting elements shall be designed to sustain, within their allowable stresses or specified strength limits, all dead loads and live loads as set out by the provisions of Sec 2.2 and 2.3 respectively. Design of roof members shall also conform to the following requirements :

- a) **Application of Loads** : When uniformly distributed loads are considered for the design of continuous structural members, load including full dead loads on all spans in combination with full live loads on adjacent spans and on alternate span, shall be investigated to determine the worst effects of loading. Concentrated roof live loads and special roof live loads, where applicable, shall also be considered in design.
- b) **Unbalanced Loading** : Effects due to unbalanced loads shall be considered in the design of roof members and connections where such loading will result in more critical stresses. Trusses and arches shall be designed to resist the stresses caused by uniform live loads on one half of the span if such loading results in reverse stresses, or stresses greater in any portion than the stresses produced by this unit live load when applied upon the entire span.
- c) **Rain Loads** : Roofs, where ponding of rain water is anticipated due to blockage of roof drains, excessive deflection or insufficient slopes, shall be designed to support such loads. Loads on roofs due to rain shall be determined in accordance with the provisions of Sec 2.6.3. In addition to the dead load of the roof, either the roof live load or the rain load, whichever is of higher intensity, shall be considered in design.

1.4.4 REDUCTION OF LIVE LOADS

The design live loads specified in Sec 2.3, may be reduced to appropriate values as permitted by the provisions of Sec 2.3.13. and Sec. 2.3.14.

1.4.5 POSTING OF LIVE LOADS

In every building, of which the floors or parts thereof have a design live load of 3.5 kN/m^2 or more, and which are used as library stack room, file room, parking garage, machine or plant room, or used for industrial or storage purposes, the owner of the building shall ensure that the live loads for which such space has been designed, are posted on durable metal plates as shown in Fig 1.1, securely affixed in a conspicuous place in each space to which they relate. If such plates are lost, removed, or defaced, the owner shall be responsible to have them replaced.

1.4.6 RESTRICTIONS ON LOADING

The building owner shall ensure that the live load for which a floor or roof is or has been designed, will not be exceeded during its use.

1.4.7 SPECIAL CONSIDERATIONS

In the absence of actual dead and live load data, the minimum values of these loads shall be those specified in Sec 2.2 and 2.3. In addition, special consideration shall be given to the following aspects of loading and due allowances shall be made in design if occurrence of such loading is anticipated after construction of a building:

- a) **Increase in Dead Load**: Actual thickness of the concrete slabs or other members may become larger than the designed thickness due to movements or deflections of the formwork during construction.

- b) Future Installations: Changes in the numbers, types and positions of partitions and other installations may increase actual load on the floors of a building.
- c) Occupancy Changes: Increase in live loads due to changes of occupancy involving loads heavier than that being designed for.

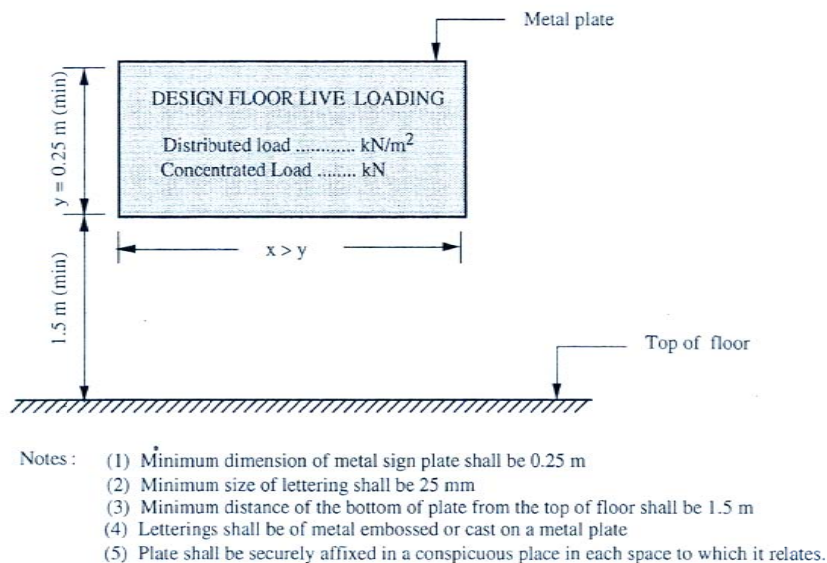


Figure 1.1: Sample Live Load Sign

1.4.8 DEFLECTION AND CAMBER

Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections. The deflections of structural members shall not exceed the more restrictive of the limitations of Chapters 2 through 13 or that permitted by Table 1.2.2. or provisions of Sec 1.2.5. In calculating deflections due to gravity loads, long term effects (e.g. creep, shrinkage or stress relaxation) should also be considered.

1.5 DESIGN FOR LATERAL LOADS

1.5.1 GENERAL

Every building, structure or portions thereof shall be designed to resist the lateral load effects, such as those due to wind or earthquake forces, in compliance with the requirements prescribed in this section.

1.5.2 SELECTION OF LATERAL FORCE FOR DESIGN

Any of the lateral loads prescribed in Chapter 2, considered either alone or in combination with other forces, whichever produces the most critical effect, shall govern the design. However, the structural detailing requirements shall comply with those prescribed in Sec 1.7. When a dual structural system is used to resist lateral loads, design shall also conform to Sec 1.3.2.4.

1.5.3 DESIGN FOR WIND LOAD

Design of buildings and their components to resist wind induced forces shall comply with the following requirements:

1.5.3.1 DIRECTION OF WIND

Structural design for wind forces shall be based on the assumption that wind may blow from any horizontal direction.

1.5.3.2 DESIGN CONSIDERATIONS

Design wind load on the primary framing systems and components of a building or structure shall be determined on the basis of the procedures provided in Sec 2.4 considering the basic wind speed, shape and size of the building, and the terrain exposure condition of the site. For slender buildings and structures, dynamic response characteristics, such as fundamental natural frequency, shall be determined for calculating the gust response coefficient. Load effects, such as forces, moments, deflections etc. on various components of the building due to wind shall be determined from a static analysis of the structure as specified in Sec 1.2.7.1.

1.5.3.3 SHIELDING EFFECT

Reductions in wind pressure on buildings and structures due to apparent direct shielding effects of the up wind obstructions, such as man-made constructions or natural terrain features, shall not be permitted.

1.5.3.4 DYNAMIC EFFECTS

Dynamic wind forces such as that from along-wind vibrations caused by the dynamic wind-structure interaction effects, as set forth by the provisions of Sec 2.4.10, shall be considered in the design of regular shaped slender buildings. For other dynamic effects such as cross-wind or torsional responses as may be experienced by buildings or structures having unusual geometrical shapes (i.e. vertical or plan irregularities listed in Tables 1.3.2 and 1.3.3), response characteristics, or site locations, structural design shall be made based on the information obtained either from other reliable references or from wind-tunnel test specified in Sec 1.5.3.5 below, complying with the other requirements of this section.

1.5.3.5 WIND TUNNEL TEST

Properly conducted wind-tunnel tests shall be required for those buildings or structures having unusual geometric shapes, response characteristics, or site locations for which cross-wind response such as vortex shedding, galloping etc. warrant special consideration, and for which no reliable literature for the determination of such effects is available. This test is also recommended for those buildings or structures for which more accurate wind-loading information is desired than those given in this section and in Sec 2.4.

Tests for the determination of mean and fluctuating components of forces and pressures shall be considered to be properly conducted only if the following requirements are satisfied:

- a) The natural wind has been modelled to account for the variation of wind speed with height,
- b) The intensity of the longitudinal components of turbulence has been taken into consideration in the model,
- c) The geometric scale of the structural model is not more than three times the geometric scale of the longitudinal component of turbulence,
- d) The response characteristics of the wind tunnel instrumentation are consistent with the measurements to be made, and
- e) The Reynolds number is taken into consideration when determining forces and pressures on the structural elements.

Tests for the purpose of determining the dynamic response of a structure shall be considered to be properly conducted only if requirements (a) through (e) above are fulfilled and, in addition, the structural model is scaled with due consideration to length, distribution of mass, stiffness and damping of the structure.

1.5.3.6 WIND LOADS DURING CONSTRUCTION

Buildings, structures and portions thereof under construction, and construction structures such as formwork, staging etc. shall be provided with adequate temporary bracings or other lateral supports to resist the wind load on them during the erection and construction phase.

1.5.3.7 MASONRY CONSTRUCTION IN HIGH-WIND REGIONS

Design and construction of masonry structures in high-wind regions shall conform to the requirements of relevant sections of Chapter 7.

1.5.3.8 HEIGHT LIMITS

Unless otherwise specified elsewhere in this Code, no height limits shall be imposed, in general, on the design and construction of buildings or structures to resist wind induced forces.

1.5.4 DESIGN FOR EARTHQUAKE FORCES

Design of structures and components thereof to resist the effects of earthquake forces shall comply with the requirements of this section.

1.5.4.1 BASIC DESIGN CONSIDERATION

For the purpose of earthquake resistant design, each structure shall be placed in one of the seismic zones as given in Sec 2.5.6.2 and assigned with a structure importance category as set forth in Sec 2.5.7.1. The seismic forces on structures shall be determined considering seismic zoning, site soil characteristics, structure importance, structural systems and configurations, height and dynamic properties of the structure as provided in Sec 2.5. The structural system and configuration types for a building or a structure shall be determined in accordance with the provisions of Sec 2.5.7.4. Other seismic design requirements shall be those specified in this section.

1.5.4.2 REQUIREMENTS FOR DIRECTIONAL EFFECTS

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. Earthquake forces act in both principal directions of the building simultaneously. Design provisions for considering earthquake component in orthogonal directions have been provided in Sec 2.5.15.1.

1.5.4.3 STRUCTURAL SYSTEM AND CONFIGURATION REQUIREMENTS

Seismic design provisions impose the following limitations on the use of structural systems and configurations:

- a) The structural system used shall satisfy requirements of the Seismic Design Category (defined in Sec. 2.5.7.2) and height limitations given in Sec 2.5.7.4.
- b) Structures assigned to Seismic Design Category D having vertical irregularity Type Vb of Table 1.3.2 shall not be permitted. Structures with such vertical irregularity may be permitted for Seismic Design Category B or C but shall not be over two stories or 9 m in height.
- c) Structures having irregular features described in Table 1.3.2 or 1.3.3 shall be designed in compliance with the additional requirements of the sections referenced in these Tables.

- d) Special Structural Systems defined in Sec 1.3.2.5 may be permitted if it can be demonstrated by analytical and test data to be equivalent, with regard to dynamic characteristics, lateral force resistance and energy absorption, to one of the structural systems listed in Table 2.5.7, for obtaining an equivalent R and C_d value for seismic design.

1.5.4.4 METHODS OF ANALYSIS

Earthquake forces and their effects on various structural elements shall be determined by using either a static analysis method or a dynamic analysis method whichever is applicable based on the limitations set forth in Sec 2.5.7 through 2.5.14 and conforming to Sec 1.2.7.

1.5.4.5 MINIMUM DESIGN SEISMIC FORCE

The minimum design seismic forces shall be those determined in accordance with the Sec 2.5.7 through 2.5.14 whichever is applicable.

1.5.4.6 DISTRIBUTION OF SEISMIC FORCES

The total lateral seismic forces and moments shall be distributed among various resisting elements at any level and along the vertical direction of a building or structure in accordance with the provisions of Sec 2.5.7 through 2.5.14 as appropriate.

1.5.4.7 VERTICAL COMPONENTS OF SEISMIC FORCES

Design provisions for considering vertical component of earthquake ground motion is given in Sec 2.5.15.2

1.5.4.8 HEIGHT LIMITS

Height limitations for different structural systems are given in Table 2.5.7 of Sec 2.5.7.4 of Part 6 of this code as a function of seismic design category.

1.5.4.9 NON-BUILDING STRUCTURES

Seismic lateral force on non-building structures shall be determined in accordance with the provisions of Chapter 15 of ASCE 7-05. However, provisions of Chapter 15 of ASCE 7-05 may be simplified, consistent with the provisions of Section 2.5 of Part 6 of this code. Other design requirements shall be those provided in this chapter.

1.5.5 OVERTURNING REQUIREMENTS

Every structure shall be designed to resist the overturning effects caused by wind or earthquake forces specified in Sec 2.4 and 2.5 respectively as well other lateral forces like earth pressure, tidal surge etc. The overturning moment M_x at any storey level-x of a building shall be determined as :

$$M_x = \sum_{i=1}^n F_i (h_i - h_x) \quad (1.5.1)$$

where,

h_i, h_x, h_n = Height in metres at level- i, -x or -n respectively.

F_i = Lateral force applied at level-i, i=1 to n.

At any level, the increment of overturning moment shall be distributed to the various resisting elements in the same manner as the distribution of horizontal shear prescribed in Sec 2.5.9.5. Overturning effects on every element shall be carried down to the foundation level.

1.5.6 DRIFT AND BUILDING SEPARATION

1.5.6.1 STOREY DRIFT LIMITATION

Storey drift is the horizontal displacement of one level of a building or structure relative to the level above or below due to the design gravity (dead and live loads) or lateral forces (e.g. wind and earthquake loads). Except otherwise permitted in Sec 1.3.4.2.1 calculated storey drift shall include both translational and torsional deflections and conform to the following requirements:

1. Storey drift, Δ , for loads other than earthquake loads, shall be limited as follows:

$$\Delta \leq 0.005h \quad \text{for } T < 0.7 \text{ second.}$$

$$\Delta \leq 0.004h \quad \text{for } T \geq 0.7 \text{ second.}$$

$$\Delta \leq 0.0025h \quad \text{for unreinforced masonry structures.}$$

where, h = height of the building or structure.

The period T used in this calculation shall be the same as that used for determining the base shear in Sec 2.5.9.2.

2. The drift limits set out in (1) above may be exceeded where it can be demonstrated that greater drift can be tolerated by both structural and nonstructural elements without affecting life safety.
3. For earthquake loads, the story drift, Δ , shall be limited in accordance with the limits set forth in Sec 2.5.16.1

1.5.6.2 SWAY LIMITATION

The overall sway (horizontal deflection) at the top level of the building or structure due to wind loading shall be limited to $1/500$ times of the total height of the building above base.

1.5.7 BUILDING SEPARATION

All components of a structure shall be designed and constructed to act as an integral unit unless they are separated structurally by a distance sufficient to avoid contact under the most unfavourable condition of deflections due to lateral loads. For seismic loads, design guidelines are given in Sec 2.5.16.3.

1.5.8 P-DELTA EFFECTS

The resulting member forces and moments and the storey drifts induced by P-Delta effects need not be considered when the stability coefficient (θ) remains within 0.10. This coefficient (described in Sec 2.5.9.9) may be evaluated for any storey as the product of the total vertical dead and live loads above the storey and the lateral drift in that storey divided by the product of the storey shear in that storey and the height of that storey.

1.5.9 UPLIFT EFFECTS

Uplift effects caused due to lateral loads shall be considered in design. When allowable (working) stress method is used for design, dead loads used to reduce uplift shall be multiplied by a factor of 0.85.

1.6 DESIGN FOR MISCELLANEOUS LOADS

1.6.1 GENERAL

Buildings, structures and components thereof, when subject to loads other than dead, live, wind and earthquake loads, shall be designed in accordance with the provisions of this section. Miscellaneous loads, such as those due to temperature, rain, flood and surge etc. on buildings or structures, shall be determined in accordance with Sec 2.6. Structural members subject to miscellaneous loads, not specified in Sec 2.6 shall be designed using well established methods given in any reliable references, and complying with the other requirements of this Code.

1.6.2 SELF-STRAINING FORCES

Self-straining forces such as those arising due to assumed differential settlements of foundations and from restrained dimensional changes due to temperature, moisture, shrinkage, creep, and similar effects, shall be taken into consideration in the design of structural members.

1.6.3 STRESS REVERSAL AND FATIGUE

Structural members and joints shall be investigated and designed against possible stress reversals caused due to various construction loads. Where required, allowance shall be made in the design to account for the effects of fatigue. The allowable stress may be appropriately reduced to account for such effects in the structural members.

1.6.4 FLOOD, TIDAL/STORM SURGE AND TSUNAMI

Buildings, structures and components thereof shall be designed, constructed and anchored to resist flotation, collapse or any permanent movement due to loads including flood, tidal/Storm surge and tsunami, when applicable. Structural members shall be designed to resist both hydrostatic and significant hydrodynamic loads and effects of buoyancy resulting from flood or surge. Flood and surge loads on buildings and structures shall be determined in accordance with Sec 2.6.4. Load combination including flood and surge loads shall conform to Sec 2.7. Design of foundations to sustain these load effects shall conform to the provisions of Sec 1.8.

Stability against overturning and sliding caused due to wind and flood or surge loads simultaneously shall be investigated, and such effects shall be resisted with a minimum factor of safety of 1.5, considering dead load only.

1.6.5 RAIN LOADS

Roofs of the buildings and structures as well as their other components which may have the capability of retaining rainwater shall be designed for adequate gravity load induced by ponding. Roofs and such other components shall be analysed and designed for load due to ponding caused by accidental blockage of drainage system complying with Sec. 2.6.3.

1.6.6 OTHER LOADS

Buildings and structures and their components shall be analyzed and designed for stresses caused by the following effects

- a. Temperature Effects (Sec 2.6.5).
- b. Soil and Hydrostatic Pressure (Sec 2.6.6).
- c. Impacts and Collisions
- d. Explosions (Sec 2.6.7).

- e. Fire
- f. Vertical Forces on Air Raid Shelters (Sec 2.6.8).
- g. Loads on Helicopter Landing Areas (Sec 2.6.9).
- h. Erection and Construction Loads (Sec 2.6.10).
- i. Moving Loads for Crane Movements
- j. Creep and Shrinkage
- k. Dynamic Loads due to Vibrations
- l. Construction Loads

Design of buildings and structures shall include loading and stresses caused by the above effects in accordance with the provisions set forth in Chapter 2.

1.7 DETAILED DESIGN REQUIREMENTS

1.7.1 GENERAL

All structural framing systems shall comply with the requirements of this section. Only the elements of the designated lateral force resisting systems can be used to resist design lateral forces specified in Chapter 2. The individual components shall be designed to resist the prescribed forces acting on them. Design of components shall also comply with the specific requirements for the materials contained in Chapters 4 through 13. In addition, such framing systems and components shall comply with the design requirements provided in this section.

1.7.2 STRUCTURAL FRAMING SYSTEMS

The basic structural systems are defined in Sec 1.3.2 and shown in Table 1.3.1, and each type is subdivided by the types of framing elements used to resist the lateral forces. The structural system used shall satisfy requirements of seismic design category and height limitations indicated in Table 2.5.7. Special framing requirements are given in the following sections in addition to those provided in Chapters 4 through 13.

1.7.3 DETAILING REQUIREMENTS FOR COMBINATIONS OF STRUCTURAL SYSTEMS :

For components common to different structural systems, a more restrictive detailing shall be provided.

1.7.3.1 CONNECTIONS TO RESIST SEISMIC FORCES

Connections which resist prescribed seismic forces shall be designed in accordance with the seismic design requirements provided in Chapters 4 through 13. Detailed sketches for these connections shall be given in the structural drawings.

1.7.3.2 DEFORMATION COMPATIBILITY

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 when subjected to lateral displacements resulting from the seismic lateral forces. For designs using working stress methods, this capacity may be determined using an allowable stress increase of 30 per cent. *P-Delta* effects on such elements shall be accounted for.

- a. **Adjoining Rigid Elements** : Moment resisting frames may be enclosed or adjoined by more rigid elements which would tend to prevent a 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.
- b. **Exterior Elements** : Exterior nonbearing, non-shearwall panels or elements which are attached to or enclose the exterior of a structure, shall be designed to resist the forces according to Sec. 2.5.17 of Chapter 2, if seismic forces are present, and shall accommodate movements of the structure resulting from lateral forces or temperature changes. Such elements shall be supported by structural members or by mechanical connections and fasteners joining them to structural members in accordance with the following provisions:
 - i. Connections and panel joints shall allow for a relative movement between storeys of not less than two times the storey drift caused by wind forces or design seismic forces, or 12 mm, whichever is greater.
 - ii. Connections to permit movement in the plane of the panel for storey drift shall be either sliding connections using slotted or oversized holes, connections which permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.
 - iii. Bodies of connections shall have sufficient ductility and rotation capability to preclude any fracture of the anchoring elements or brittle failures at or near weldings.
 - iv. Bodies of the connection shall be designed for 1.33 times the seismic force determined by Sec. 2.5.17 of Chapter 2, or equivalent.
 - v. All fasteners in the connection system, such as bolts, inserts, welds, dowels etc. shall be designed for 4 times the forces determined by Sec. 2.5.17 of Chapter 2 or equivalent.
 - vi. Fasteners embedded in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to transfer forces to the reinforcing steel effectively.

1.7.3.3 TIES AND CONTINUITY

All parts of a structure shall be interconnected. These connections shall be capable of transmitting the prescribed lateral force to the lateral force resisting system. Individual members, including those not part of the seismic force-resisting system, shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with this standard. Connections shall develop the strength of the connected members and shall be capable of transmitting the seismic force (F_p) induced by the parts being connected.

1.7.3.4 COLLECTOR ELEMENTS

Collector elements shall be provided which are capable of transferring the lateral forces originating in other portions of the structure to the element providing the resistance to those forces.

1.7.3.5 CONCRETE FRAMES

When concrete frames are provided by design to be part of the lateral force resisting system, they shall conform to the following provisions:

- a) In Seismic Zones 3 and 4 these frames shall be designed as special moment resisting frames (SMRF).
- b) In Seismic Zone 2 they shall, as a minimum, be intermediate moment resisting frames (IMRF).

1.7.3.6 ANCHORAGE OF CONCRETE AND MASONRY STRUCTURAL WALLS

The concrete and masonry structural walls shall be anchored to supporting construction. The anchorage shall provide a positive direct connection between the wall and floor or roof and shall be capable of resisting the horizontal forces specified in Secs 2.4.13 and 2.5.17, or a minimum force of 4.09 kN/m of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 m. In masonry walls of hollow units or cavity walls, anchors shall be embedded in a reinforced grouted structural element of the wall. Deformations of the floor and roof diaphragms shall be considered in the design of the supported walls and the anchorage forces in the diaphragms shall be determined in accordance with Sec 1.7.3.9 below.

1.7.3.7 BOUNDARY MEMBERS

Specially detailed boundary members shall be considered for shearwalls and shearwall elements whenever their design is governed by flexure.

1.7.3.8 FLOOR AND ROOF DIAPHRAGMS

Deflection in the plane of the diaphragm 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. Design of diaphragms shall also comply with the following requirements.

- a) **Diaphragm Forces** : Diaphragms shall be designed to resist the seismic forces given in Sec 2.5 or for similar non-seismic lateral forces, whichever is greater.
- b) **Diaphragm Ties** : Diaphragms supporting concrete or masonry walls shall have continuous ties, or struts between the diaphragm chords to distribute the anchorage forces specified in Sec 1.7.3.6 above. Added chords may be provided to form sub-diaphragms to transmit the anchorage forces to the main cross ties.
- c) **Wood Diaphragms** : Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Sec 1.7.3.6 above. In seismic Zones 2, 3 and 4 the following requirements shall also apply:
 - i. Anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension.
 - ii. The continuous ties required by paragraph (b) above, shall be in addition to the diaphragm sheathing.
- d) **Structures having irregularities**
 - i) For structures assigned to Seismic Design Category D and having a plan irregularity of Type I, II, III, or IV in Table 1.3.3 or a vertical structural irregularity of Type IV in Table 1.3.2, the design forces determined from Section 2.5.9 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor.
 - ii) For structures having a plan irregularity of Type II in Table 1.3.3, diaphragm chords and collectors shall be designed considering independent movement of any projecting wings of

the structure. Each of these diaphragm elements shall be designed for the more severe of the following cases:

1. Motion of the projecting wings in the same direction.
2. Motion of the projecting wings in opposing directions.

Exception:

This requirement may be deemed to be satisfied if the procedures of Sec 2.5.10 when seismic forces are present, in conjunction with a three dimensional model, have been used to determine the lateral seismic forces for design.

1.7.3.9 FRAMING BELOW THE BASE

When structural framings continue below the base, the following requirements shall be satisfied.

- a. Framing between the Base and the Foundation: The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure . The special detailing requirements of Sec 8.3 or 10.20, as appropriate for reinforced concrete or steel, shall apply to columns supporting discontinuous lateral force resisting elements and to SMRF, IMRF, and EBF system elements below the base which are required to transmit the forces resulting from lateral loads to the foundation.
- b. Foundations: The foundation shall be capable of transmitting the design base shear and the overturning forces from the superstructure into the supporting soil, but the short term dynamic nature of the loads may be taken into account in establishing the soil properties. Sec 1.8 below prescribes the additional requirements for specific types of foundation construction.

1.8 FOUNDATION DESIGN REQUIREMENTS

1.8.1 GENERAL

The design and construction of foundation, foundation components and connection between the foundation and superstructure shall conform to the requirements of this section and applicable provisions of Chapter 3 and other portions of this Code .

1.8.2 SOIL CAPACITIES

The bearing capacity of the soil, or the capacity of the soil-foundation system including footing, pile, pier or caisson and the soil, shall be sufficient to support the structure with all prescribed loads, considering the settlement of the structure. For piles, this refers to pile capacity as determined by pile-soil friction and bearing which may be determined in accordance with the provisions of Chapter 3. For the load combination including earthquake, the soil capacity shall be sufficient to resist loads at acceptable strains considering both the short time loading and the dynamic properties of the soil. The stress and settlement of soil under applied loads shall be determined based on established methods of Soil Mechanics.

1.8.3 SUPERSTRUCTURE-TO-FOUNDATION CONNECTION

The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements are required to be designed.

1.8.4 FOUNDATION-SOIL INTERFACE

For regular buildings the base overturning moments for the entire structure or for any one of its lateral force-resisting elements, shall not exceed two-thirds of the dead load resisting moment. The weight of the earth superimposed over footings may be used to calculate the dead load resisting moment.

1.8.5 SPECIAL REQUIREMENTS FOR FOOTINGS, PILES AND CAISSONS IN SEISMIC ZONES 2, 3 AND 4

1.8.5.1 PILES AND CAISSONS

Piles and caissons shall be designed for flexure whenever the top of such members is anticipated to be laterally displaced by earthquake motions. The criteria and detailing requirements of Sec 8.3 for concrete and Sec 10.20 for steel shall apply for a length of such members equal to 120 per cent of the flexural length.

1.8.5.2 FOOTING INTERCONNECTION

- a. Footings and pile caps shall be completely interconnected by strut ties or other equivalent means to restrain their lateral movements in any orthogonal direction.
- b. The strut ties or other equivalent means as specified in (a) above, shall be capable of resisting in tension or compression a force not less than 10% of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by frictional and passive soil resistance or by other established means.

1.8.6 RETAINING WALL DESIGN

Retaining walls shall be designed to resist the lateral pressure of the retained material, under drained or undrained conditions and including surcharge, in accordance with established engineering practice. For such walls, the minimum factor of safety against base overturning and sliding due to applied earth pressure shall be 1.5.

1.9 DESIGN AND CONSTRUCTION REVIEW

Every building or structure designed shall have its design documents prepared in accordance with the provisions of Sec 1.9.1. The minimum requirements for design review and construction observation shall be those set forth under Sec 1.9.2 and 1.9.3 respectively.

1.9.1 DESIGN DOCUMENT

The design documents shall be prepared and signed by the engineer responsible for the structural design of any building or structure intended for construction. The design documents shall include a design report, material specifications and a set of structural drawings, which shall be prepared in compliance with Sec 1.9.2 and 1.9.3 below for submittal to the concerned authority. For the purpose of this provision, the concerned authority shall be either persons from the government approval agency for the construction, or the owner of the building or the structure, or one of his representatives.

1.9.2 DESIGN REPORT

The design report shall contain the description of the structural design with basic design information as provided below, so that any other structural design engineer will be able to independently verify the design parameters and the member sizes using these basic information. The design report shall include, but not be limited to, the following :

- a) Name and governing edition of this Code and other referenced standards, and the specific portions, stating chapter, section, clause etc. of these Code and standards including any specialist report used for the structural design.
- b) Methods used for the calculation of all applied loads along with basic load coefficients and other basic information including any assumption or judgment made under special circumstances.
- c) A drawing of the complete mathematical model prepared in accordance with Sec 1.2.7.1 to represent the structure and showing on it the values, locations and directions of all applied loads, and location of the lateral load resisting systems such as shear walls, braced frames etc.
- d) Methods of structural analysis, and results of the analysis such as shear, moment, axial force etc., used for proportioning various structural members and joints including foundation members.
- e) Methods of structural design including types and strength of the materials of construction used for proportioning the structural members.
- f) Reference of the soil report or any other documents used in the design of the structure, foundation or components thereof.
- g) Statement supporting the validity of the above design documents with date and signature of the engineer responsible for the structural design.
- h) When computer programs are used, to any extent, to aid in the analysis or design of the structure, the following items, in addition to items (a) through (g) above, shall be required to be included in the design report:
 - i. A sketch of the mathematical model used to represent the structure in the computer generated analysis.
 - ii. The computer output containing the date of processing, program identification, identification of structures being analysed, all input data, units and final results. The computer input data shall be clearly distinguished from those computed in the program.
 - iii. A program description containing the information necessary to verify the input data and interpret the results to determine the nature and extent of the analysis and to check whether the computations comply with the provisions of this Code.
 - iv. The first sheet of each computer run shall be signed by the engineer responsible for the structural design.

1.9.3 STRUCTURAL DRAWINGS AND MATERIAL SPECIFICATIONS

The structural drawings shall include, but not be limited to, the following :

- a) The first sheet shall contain : (1) identification of the project to which the building or the structure, or portion thereof belongs, (2) reference to the design report specified in Sec 1.9.2 above, (3) date of completion of design, and (4) identification and signature with date of the engineer responsible for the structural design.

- b) The second sheet shall contain detail material specifications showing : (1) Specified compressive strength of concrete at stated ages or stages of construction for which each part of structure is designed (2) Specified strength or grade of reinforcement (3) Specified strength of prestressing tendons or wires (4) Specified strength or grade of steel (5) Specified strengths for bolts, welds etc. (6) Specified strength of masonry, timber, bamboo, ferrocement (7) Minimum concrete compressive strength at time of post-tensioning (8) Stressing sequence for post-tensioning tendons (9) General notes indicating clear cover, development lengths of reinforcements, or any other design parameter relevant to the member or connection details provided in drawings to be followed, as applicable, and (10) identification and signature with date of the engineer responsible for the structural design.
- c) Drawing sheets, other than the first two, shall include structural details of the elements of the structure clearly showing all sizes, cross-sections and relative locations, connections, reinforcements, laps, stiffeners, welding types, lengths and locations etc. whichever is applicable for a particular construction. Floor levels, column centres and offset etc., shall be dimensioned. Camber of trusses and beams, if required, shall be shown on drawings. For bolt connected members, connection types such as slip, critical, tension or bearing type, shall be indicated on the drawing.
- d) Drawings shall be prepared to a scale large enough to show the information clearly and the scales shall be marked on the drawing sheets. If any variation from the design specifications provided in sheet two occurs, the drawing sheet shall be provided additionally with the design specifications including material types and strength, clear cover and development lengths of reinforcements, or any other design parameter relevant to the member or connection details provided in that drawing sheet. Each drawing sheet shall also contain the signature with date of the engineer responsible for the structural design.

1.9.4 DESIGN REVIEW

The design documents specified in Sec 1.9.1 shall be available for review when required by the concerned authority. Review shall be accomplished by an independent structural engineer qualified for this task and appointed by the concerned authority. Design review shall be performed through independent calculations, based on the information provided in the design documents prepared and signed by the original structural design engineer, to verify the design parameters including applied loads, methods of analysis and design, and final design dimensions and other details of the structural elements. The reviewing engineer shall also check the sufficiency and appropriateness of the supplied structural drawings for construction.

1.9.5 CONSTRUCTION OBSERVATION

Construction observation shall be performed by a responsible person who will be a competent professional appointed by the owner of the building or the structure. Construction observation shall include, but not be limited to, the following :

- a. Specification of an appropriate testing and inspection schedule prepared and signed with date by the responsible person;
- b. Review of testing and inspection reports; and
- c. Regular site visit to verify the general compliance of the construction work with the structural drawings and specifications provided in Sec 1.9.3 above.

LOADS ON BUILDINGS AND STRUCTURES

2.1 INTRODUCTION

2.1.1 SCOPE

This chapter specifies the minimum design forces including dead load, live load, wind and earthquake loads, miscellaneous loads and their various combinations. These loads shall be applicable for the design of buildings and structures in conformance with the general design requirements provided in Chapter 1.

2.1.2 LIMITATIONS

Provisions of this chapter shall generally be applied to majority of buildings and other structures covered in this code subject to normally expected loading conditions. For those buildings and structures having unusual geometrical shapes, response characteristics or site locations, or for those subject to special loading including tornadoes, special dynamic or hydrodynamic loads etc., site-specific or case-specific data or analysis may be required to determine the design loads on them. In such cases, and all other cases for which loads are not specified in this chapter, loading information may be obtained from reliable references or specialist advice may be sought. However, such loads shall be applied in compliance with the provisions of other parts or sections of this Code.

2.2 DEAD LOADS

2.2.1 GENERAL

The minimum design dead load for buildings and portions thereof shall be determined in accordance with the provisions of this section. In addition, design of the overall structure and its primary load-resisting systems shall conform to the general design provisions given in Chapter 1.

2.2.2 DEFINITION

Dead Load is the vertical load due to the weight of permanent structural and non-structural components and attachments of a building such as walls, floors, ceilings, permanent partitions and fixed service equipment etc.

2.2.3 ASSESSMENT OF DEAD LOAD

Dead load for a structural member shall be assessed based on the forces due to :

- weight of the member itself,
- weight of all materials of construction incorporated into the building to be supported permanently by the member,
- weight of permanent partitions,
- weight of fixed service equipment, and
- net effect of prestressing.

2.2.4 WEIGHT OF MATERIALS AND CONSTRUCTIONS

In estimating dead loads, the actual weights of materials and constructions shall be used, provided that in the absence of definite information, the weights given in Tables 2.2.1 and 2.2.2 shall be assumed for the purposes of design.

Table 2.2.1 : Unit Weight of Basic Materials

Material	Unit weight (kN/m ³)	Material	Unit Weight (kN/m ³)
Aluminium	27.0	Granite, Basalt	26.4
Asphalt	21.2	Iron - cast	70.7
Brass	83.6	- wrought	75.4
Bronze	87.7	Lead	111.0
Brick	18.9	Limestone	24.5
Cement	14.7	Marble	26.4
Coal, loose	8.8	Sand, dry	15.7
Concrete - stone aggregate (unreinforced)	22.8*	Sandstone	22.6
- brick aggregate (unreinforced)	20.4*	Slate	28.3
Copper	86.4	Steel	77.0
Cork, normal	1.7	Stainless Steel	78.75
Cork, compressed	3.7	Timber	5.9-11.0
Glass, window (soda-lime)	25.5	Zinc	70.0

* for reinforced concrete, add 0.63 kN/m³ for each 1% by volume of main reinforcement

2.2.5 WEIGHT OF PERMANENT PARTITIONS

When partition walls are indicated on the plans, their weight shall be considered as dead load acting as concentrated line loads in their actual positions on the floor. The loads due to anticipated partition walls, which are not indicated on the plans, shall be treated as live loads and determined in accordance with Sec 2.3.2.4.

2.2.6 WEIGHT OF FIXED SERVICE EQUIPMENT

Weights of fixed service equipment and other permanent machinery, such as electrical feeders and other machinery, heating, ventilating and air-conditioning systems, lifts and escalators, plumbing stacks and risers etc. shall be included as dead load whenever such equipment are supported by structural members.

2.2.7 ADDITIONAL LOADS

In evaluating the final dead loads on a structural member for design purposes, allowances shall be made for additional loads resulting from the (i) difference between the prescribed and the actual weights of the members and construction materials; (ii) inclusion of future installations; (iii) changes in occupancy or use of buildings; and (iv) inclusion of structural and non-structural members not covered in Sec 2.2.2 and 2.2.3.

2.3 LIVE LOADS

2.3.1 GENERAL

The live loads used for the structural design of floors, roof and the supporting members shall be the greatest applied loads arising from the intended use or occupancy of the building, or from the stacking of materials and the use of equipment and propping during construction, but shall not be less than the minimum design live loads set out by the provisions of this section. For the design of structural members for forces including live loads, requirements of the relevant sections of Chapter 1 shall also be fulfilled.

Table 2.2.2 Dead Load

Material / Component / Member	Weight per Unit Area (kN/m ²)	Material	Weight per Unit Area (kN/m ²)
Floor		Walls and Partitions	
Asphalt, 25 mm thick	0.526	Acrylic resin sheet, flat, per mm thickness	0.012
Clay tiling, 13 mm thick	0.268	Asbestos cement sheeting ----	0.072
Concrete slab (stone aggregate)* ---		4.5 mm thick	0.106
solid, 100 mm thick	2.360	6.0 mm thick	
solid, 150 mm thick	3.540	Brick masonry work, excl. plaster ---	1.910
Galvanized steel floor deck (excl. topping)	0.147-0.383	burnt clay, per 100 mm thickness	1.980
Magnesium oxychloride-		sand-lime, per 100 mm thickness	2.360
normal (sawdust filler), 25 mm thick	0.345	Concrete (stone aggregate)* ---	3.540
heavy duty (mineral filler), 25 mm thick	0.527	100 mm thick	5.900
Terrazzo paving 16 mm thick	0.431	150 mm thick	0.034
		250 mm thick	0.092
Roof		Fibre insulation board, per 10 mm thickness	0.269
Acrylic resin sheet, corrugated ----		thickness	0.961
3 mm thick, standard corrugations	0.043	Fibrous plaster board, per 10 mm thickness	0.075
3 mm thick, deep corrugations	0.062	thickness	0.092
Asbestos cement, corrugated sheeting ---		Glass, per 10 mm thickness	0.061
(incl. lap and fastenings)		Hardboard, per 10 mm thickness	
6 mm thick (standard corrugations)	0.134	Particle or flake board, per 10 mm thickness	0.081
6 mm thick(deep corrugations)	0.158	thickness	0.287
Aluminium, corrugated sheeting ---		Plaster board, per 10 mm thickness	0.480
(incl. lap and fastenings)		Plywood, per 10 mm thickness	
1.2 mm thick	0.048		
0.8 mm thick	0.028	Ceiling	
0.6 mm thick	0.024	Fibrous plaster, 10 mm thick	0.019
Aluminium sheet(plain) ---		Cement plaster, 13 mm thick	
1.2 mm thick	0.033	Suspended metal lath and plaster	0.230
1.0 mm thick	0.024	(two faced incl. studding)	0.191
0.8 mm thick	0.019		0.153
Bituminous felt(5 ply) and gravel	0.431	Miscellaneous	0.151
Slates ---		Felt (insulating), per 10 mm thickness	
4.7 mm thick	0.335	Plaster ---	0.671
9.5 mm thick	0.671	cement, per 10 mm thickness	0.995
Steel sheet, flat galvanized ---		lime, per 10 mm thickness	1.388
1.00 mm thick	0.082	PVC sheet, per 10 mm thickness	
0.80 mm thick	0.067	Rubber paving, per 10 mm thickness	
0.60 mm thick	0.053	Terra-cotta Hollow Block Masonry ---	
Steel, galvanized std. corrugated sheeting ---		75 mm thick	
(incl. lap and fastenings)		100 mm thick	
1.0 mm thick	0.120	150 mm thick	
0.8 mm thick	0.096		
0.6 mm thick	0.077		
Tiles ---			
terra-cotta (French pattern)	0.575		
concrete , 25 mm thick	0.527		
clay tiles	0.6-0.9		

* for brick aggregate, 90% of the listed values may be used.

2.3.2 DEFINITION

Live load is the load superimposed by the use or occupancy of the building not including the environmental loads such as wind load, rain load, earthquake load or dead load.

2.3.3 MINIMUM FLOOR LIVE LOADS

The minimum floor live loads shall be the greatest actual imposed loads resulting from the intended use or occupancy of the floor, and shall not be less than the uniformly distributed load patterns specified in Sec 2.3.4 or the concentrated loads specified in Sec 2.3.5 whichever produces the most critical effect. The live loads shall be assumed to act vertically upon the area projected on a horizontal plane.

Table 2.3.1 Minimum Uniformly Distributed Live Loads, And Minimum Concentrated Live Loads

Occupancy or Use	Uniform kN/m ²	Conc. kN
Apartments (see <i>Residential</i>)		
Access floor systems		
Office use	2.4	8.9
Computer use	4.79	8.9
Armories and drill rooms	7.18	
Assembly areas and theaters		
Fixed seats (fastened to floor)	2.87	
Lobbies	4.79	
Movable seats	4.79	
Platforms (assembly)	4.79	
Stage floors	7.18	
Balconies (exterior)	4.79	
On one- and two-family residences only, and not exceeding 19.3 m ²	2.87	
Bowling alleys, poolrooms, and similar recreational areas	3.59	
Catwalks for maintenance access	1.92	1.33
Corridors		
First floor	4.79	
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	4.79	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	4.79	
Dwellings (see <i>Residential</i>)		
Elevator machine room grating (on area of 2,580 mm ²)		1.33
Finish light floor plate construction (on area of 645 mm ²)		0.89
Fire escapes	4.79	
On single-family dwellings only	1.92	
Fixed ladders	See Section 2.3.11	
Garages (passenger vehicles only) Trucks and buses	1.92 ^{a, b}	
Grandstands (see <i>Stadiums and arenas, Bleachers</i>)		
Gymnasiums—main floors and balconies	4.79	
Handrails, guardrails, and grab bars	See Section 2.3.11	
Hospitals		
Operating rooms, laboratories	2.87	4.45
Patient rooms	1.92	4.45
Corridors above first floor	3.83	4.45
Hotels (see <i>Residential</i>)		
Libraries		
Reading rooms	2.87	4.45
Stack rooms	7.18 ^c	4.45
Corridors above first floor	3.83	4.45
Manufacturing		
Light	6.00	8.90
Heavy	11.97	13.40
Marquees	3.59	
Office Buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	4.79	8.90
Offices	2.40	8.90
Corridors above first floor	3.83	8.90

Table 2.3.1 Minimum Uniformly Distributed Live Loads, And Minimum Concentrated Live Loads (Contd.)

Penal Institutions		
Cell blocks	1.92	
Corridors	4.79	
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	0.48	
Uninhabitable attics with storage	0.96	
Habitable attics and sleeping areas	1.44	
All other areas except stairs and balconies	1.92	
Hotels and multifamily houses		
Private rooms and corridors serving them	1.92	
Public rooms and corridors serving them	4.79	
Reviewing stands, grandstands, and bleachers	4.79 ^d	
Roofs		
Ordinary flat, pitched, and curved roofs	0.96 ^h	
Roofs used for promenade purposes	2.87	
Roofs used for roof gardens or assembly purposes	4.79	
Roofs used for other special purposes	<i>i</i>	<i>i</i>
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	0.24(nonreduceable)	
All other construction	0.96	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		8.9
All other occupancies		1.33
All roof surfaces subject to maintenance workers		1.33
Schools		
Classrooms	1.92	4.45
Corridors above first floor	3.83	4.45
First-floor corridors	4.79	4.45
Scuttles, skylight ribs, and accessible ceilings		0.89
Sidewalks, vehicular driveways, and yards subject to trucking	11.97 ^e	35.60 ^f
Stadiums and arenas		
Bleachers	4.79 ^d	
Fixed seats (fastened to floor)	2.87 ^d	
Stairs and exit ways	4.79	<i>g</i>
One- and two-family residences only	1.92	
Storage areas above ceilings	0.96	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	6.00	
Heavy	11.97	
Stores		
Retail		
First floor	4.79	4.45
Upper loors	3.59	4.45
Wholesale, all floors	6.00	4.45
Vehicle barriers	See Section 2.3.11	
Walkways and elevated platforms (other than exit ways)	2.87	
Yards and terraces, pedestrian	4.79	

^a Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 2.3.1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 13.35 kN acting on an area of 114 mm by 114 mm footprint of a jack; and (2) for mechanical parking structures without slab or deck that are used for storing passenger car only, 10 kN per wheel.

- b* Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.
- c* The loading applies to stack room floors that support non-mobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 2290 mm; (2) the nominal shelf depth shall not exceed 305 mm for each face; and (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 914 mm wide.
- d* In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 0.350 kN per linear meter of seat applied in a direction parallel to each row of seats and 0.15 kN per linear meter of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.
- e* Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.
- f* The concentrated wheel load shall be applied on an area of 114 mm by 114 mm footprint of a jack.
- g* Minimum concentrated load on stair treads (on area of 2,580 mm²) is 1.33 kN.
- h* Where uniform roof live loads are reduced to less than 1.0 kN/m² in accordance with Section 2.3.14.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable effect.
- i* Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

2.3.4 UNIFORMLY DISTRIBUTED LOADS

The uniformly distributed live load shall not be less than the values listed in Table 2.3.1, reduced as may be specified in Sec 2.3.13, applied uniformly over the entire area of the floor, or any portion thereof to produce the most adverse effects in the member concerned.

2.3.5 CONCENTRATED LOADS

The concentrated load to be applied non-concurrently with the uniformly distributed load given in Sec 2.3.4, shall not be less than that listed in Table 2.3.1. Unless otherwise specified in Table 2.3.1 or in the following paragraph, the concentrated load shall be applied over an area of 300 mm x 300 mm and shall be located so as to produce the maximum stress conditions in the structural members.

In areas where vehicles are used or stored, such as car parking garages, ramps, repair shops etc., provision shall be made for concentrated loads consisting of two or more loads spaced nominally 1.5 m on centres in absence of the uniform live loads. Each load shall be 40 per cent of the gross weight of the maximum size vehicle to be accommodated and applied over an area of 750 mm x 750 mm. For the storage of private or pleasure-type vehicles without repair or fuelling, floors shall be investigated in the absence of the uniform live load, for a minimum concentrated wheel load of 9 kN spaced 1.5 m on centres, applied over an area of 750 mm x 750 mm. The uniform live loads for these cases are provided in Table 2.3.1. The condition of concentrated or uniform live load producing the greater stresses shall govern.

2.3.6 PROVISION FOR PARTITION WALLS

When partitions, not indicated on the plans, are anticipated to be placed on the floors, their weight shall be included as an additional live load acting as concentrated line loads in an arrangement producing the most severe effect on the floor, unless it can be shown that a more favourable arrangement of the partitions shall prevail during the future use of the floor.

In the case of light partitions, wherein the total weight per metre run is not greater than 5.5 kN, a uniformly distributed live load may be applied on the floor in lieu of the concentrated line loads specified above. Such uniform live load per square metre shall be at least 33% of the weight per metre run of the partitions, subject to a minimum of 1.2 kN/m².

2.3.7 MORE THAN ONE OCCUPANCY

Where an area of a floor is intended for two or more occupancies at different times, the value to be used from Table 2.3.1 shall be the greatest value for any of the occupancies concerned.

2.3.8 MINIMUM ROOF LIVE LOADS

Roof live loads shall be assumed to act vertically over the area projected by the roof or any portion of it upon a horizontal plane, and shall be determined as specified in the following sections :

2.3.8.1 REGULAR PURPOSE - FLAT, PITCHED AND CURVED ROOFS

Live loads on regular purpose roofs shall be the greatest applied loads produced during use by movable objects such as planters and people, and those induced during maintenance by workers, equipment and materials but shall not be less than those given in Table 2.3.2.

2.3.8.2 SPECIAL PURPOSE ROOFS

For special purpose roofs, live loads shall be estimated based on the actual weight depending on the type of use, but shall not be less than the following values :

- a) roofs used for promenade purposes - 3.0 kN/m²
- b) roofs used for assembly purposes - 5.0 kN/m²
- c) roofs used for gardens - 5.0 kN/m²
- d) roofs used for other special purposes- to be determined as per Sec 2.3.9

2.3.8.3 ACCESSIBLE ROOF SUPPORTING MEMBERS

Roof trusses or any other primary roof supporting member beneath which a full ceiling is not provided, shall be capable of supporting safely, in addition to other roof loads, a concentrated load at the locations as specified below :

- a) Industrial, Storage and Garage Buildings - Any single panel point of the lower chord of a roof truss, or any point of other primary roof supporting member 9.0 kN
- b) Building with Other Occupancies - Any single panel point of the lower chord of a roof truss, or any point of other primary roof supporting member 1.3 kN

2.3.9 LOADS NOT SPECIFIED

Live loads, not specified for uses or occupancies in Sec 2.3.3, 2.3.4 and 2.3.5, shall be determined from loads resulting from :

- a) weight of the probable assembly of persons;
- b) weight of the probable accumulation of equipment and furniture, and
- c) weight of the probable storage of materials.

2.3.10 PARTIAL LOADING AND OTHER LOADING ARRANGEMENTS

The full intensity of the appropriately reduced live load applied only to a portion of the length or area of a structure or member shall be considered, if it produces a more unfavourable effect than the same intensity applied over the full length or area of the structure or member.

Where uniformly distributed live loads are used in the design of continuous members and their supports, consideration shall be given to full dead load on all spans in combination with full live loads on adjacent spans and on alternate spans whichever produces a more unfavourable effect.

Table 2.3.2 : Minimum Roof Live Loads⁽¹⁾

Type and Slope of Roof		Distributed Load, kN/m ²	Concentrated Load, kN
I	Flat roof (slope = 0)	1.5	1.8
II	1. Pitched or sloped roof (0 < slope < 1/3) 2. Arched roof or dome (rise < 1/8 span)	1.0	0.9
III	1. Pitched or sloped roof (1/3 ≤ slope < 1.0) 2. Arched roof or dome (1/8 ≤ rise < 3/8 span)	0.8	0.9
IV	1. Pitched or sloped roof (slope ≥ 1.0) 2. Arched roof or dome (rise ≥ 3/8 span)	0.6	0.9
V	Greenhouse, and agriculture buildings	0.5	0.9
VI	Canopies and awnings, except those with cloth covers	same as given in I through IV above based on the type and slope.	

Note : (1) Greater of this load and rain load as specified in Sec 2.6.3 shall be taken as the design live load for roof. The distributed load shall be applied over the area of the roof projected upon a horizontal plane and shall not be applied simultaneously with the concentrated load. The concentrated load shall be assumed to act upon a 300 mm x 300 mm area and need not be considered for roofs capable of laterally distributing the load, e.g. reinforced concrete slabs.

2.3.11 OTHER LIVE LOADS

Live loads on miscellaneous structures and components, such as handrails and supporting members, parapets and balustrades, ceilings, skylights and supports, and the like, shall be determined from the analysis of the actual loads on them, but shall not be less than those given in Table 2.3.3.

2.3.12 IMPACT AND DYNAMIC LOADS

The live loads specified in Sec 2.3.3 shall be assumed to include allowances for impacts arising from normal uses only. However, forces imposed by unusual vibrations and impacts resulting from the operation of installed machinery and equipment shall be determined separately and treated as additional live loads. Live loads due to vibration or impact shall be determined by dynamic analysis of the supporting member or structure including foundations, or from the recommended values supplied by the manufacture of the particular equipment or machinery. In absence of a definite information, values listed in Table 2.3.4 for some common equipment, shall be used for design purposes.

2.3.13 REDUCTION OF LIVE LOADS

Except for roof uniform live loads, all other minimum uniformly distributed live loads, L_0 in Table 2.3.1, may be reduced according to the following provisions.

2.3.13.1 GENERAL

Subject to the limitations of Sections 2.3.13.2 through 2.3.13.5, members for which a value of $K_{LL} A_T$ is 37.16 m² or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_0 \left(0.25 + \frac{4.57}{\sqrt{K_{LL} A_T}} \right) \quad (2.3.1)$$

where, L = reduced design live load per m² of area supported by the member; L_0 = unreduced design live load per m² of area supported by the member (see Table 2.3.1); K_{LL} = live load element factor (see Table 2.3.5); A_T = tributary area in m². L shall not be less than $0.50L_0$ for members supporting one floor and L shall not be less than $0.40L_0$ for members supporting two or more floors.

Table 2.3.3 : Miscellaneous Live Loads

Structural Member or Component		Live Load ⁽¹⁾ (kN/m)
1.	Handrails, parapets and supports :	
	a) Light access stairs, gangways etc.	
	i) width ≤ 0.6 m	0.25
	ii) width > 0.6 m	0.35
	b) Staircases other than in (a) above, ramps, balconies :	
	i) Single dwelling and private	0.35
	ii) Staircases in residential buildings	0.35
	iii) Balconies or portion thereof, stands etc. having fixed seats within 0.55 m of the barrier	1.5
	vi) Public assembly buildings including theatres, cinemas, assembly halls, stadiums, mosques, churches, schools etc.	3.0
vi)	Buildings and occupancies other than (i) through (iv) above	0.75
2.	Vehicle barriers for car parks and ramps :	
	a) For vehicles having gross mass ≤ 2500 kg	100 ⁽²⁾
	b) For vehicles having gross mass > 2500 kg	165 ⁽²⁾
	c) For ramps of car parks etc.	see note (3)
Note :	(1) These loads shall be applied non-concurrently along horizontal and vertical directions, except as specified in note (2) below.	
	(2) These loads shall be applied only in the horizontal direction, uniformly distributed over any length of 1.5 m of a barrier and shall be considered to act at bumper height. For case 2(a) bumper height may be taken as 375 mm above floor level.	
	(3) Barriers to access ramps of car parks shall be designed for horizontal forces equal to 50% of those given in 2(a) and 2(b) applied at a level of 610 mm above the ramp. Barriers to straight exit ramps exceeding 20 m in length shall be designed for horizontal forces equal to twice the values given in 2(a) and 2(b).	

Table 2.3.4 : Minimum Live Loads on Supports and Connections of Equipment due to Impact ⁽¹⁾

Equipment or Machinery		Additional load due to impact as percentage of static load including self weight	
		Vertical	Horizontal
1.	Lifts, hoists and related operating machinery	100%	–
2.	Light machinery (shaft or motor driven)	20%	–
3.	Reciprocating machinery, or power driven units.	50%	–
4.	Hangers supporting floors and balconies	33%	–
5.	Cranes :		
	a) Electric overhead cranes	25% of maximum wheel load	i) Transverse to the rail : 20% of the weight of trolley and lifted load only, applied one-half at the top of each rail ii) Along the rail : 10% of maximum wheel load applied at the top of each rail
	b) Manually operated cranes	50% of the values in (a) above	50% of the values in (a) above
	c) Cab-operated travelling cranes	25%	–
–	Not applicable		
Note : (1)	All these loads shall be increased if so recommended by the manufacturer. For machinery and equipment not listed, impact loads shall be those recommended by the manufacturers, or determined by dynamic analysis.		

2.3.13.2 HEAVY LIVE LOADS.

Live loads that exceed 4.79 kN/m^2 shall not be reduced.

EXCEPTION: Live loads for members supporting two or more floors may be reduced by 20 percent.

2.3.13.3 PASSENGER CAR GARAGES.

The live loads shall not be reduced in passenger car garages.

EXCEPTION: Live loads for members supporting two or more floors may be reduced by 20 percent.

2.3.13.4 SPECIAL OCCUPANCIES.

- Live loads of 4.79 kN/m^2 or less shall not be reduced in public assembly occupancies.
- There shall be no reduction of live loads for cyclone shelters.

2.3.13.5 LIMITATIONS ON ONE-WAY SLABS.

The tributary area, A_T , for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

2.3.14 REDUCTION IN ROOF LIVE LOADS

The minimum uniformly distributed roof live loads, L_o in Table 2.3.1, are permitted to be reduced according to the following provisions.

2.3.14.1 FLAT, PITCHED, AND CURVED ROOFS.

Ordinary flat, pitched, and curved roofs are permitted to be designed for a reduced roof live load, as specified in Eq.2.3.2 or other controlling combinations of loads, as discussed later in this chapter, whichever produces the greater load. In structures such as greenhouses, where special scaffolding is used as a work surface for workmen and materials during maintenance and repair operations, a lower roof load than specified in Eq. 2.3.2 shall not be used unless approved by the authority having jurisdiction. On such structures, the minimum roof live load shall be 0.58 kN/m^2 .

$$L_r = L_o R_1 R_2 \text{ where } 0.58 \leq L_r \leq 0.96 \quad (2.3.2)$$

where

L_r = reduced roof live load per m^2 of horizontal projection in kN/m^2

The reduction factors R_1 and R_2 shall be determined as follows:

$$\begin{aligned} R_1 &= 1 \text{ for } A_t \leq 18.58 \text{m}^2 \\ &= 1.2 - 0.011A_t \text{ for } 18.58 \text{m}^2 < A_t < 55.74 \text{m}^2 \\ &= 0.6 \text{ for } A_t \geq 55.74 \text{m}^2 \end{aligned}$$

where A_t = tributary area in m^2 supported by any structural member and

$$\begin{aligned} R_2 &= 1 \text{ for } F \leq 4 \\ &= 1.2 - 0.05F \text{ for } 4 < F < 12 \\ &= 0.6 \text{ for } F \geq 12 \end{aligned}$$

where, for a pitched roof, $F = 0.12 \times \text{slope}$, with slope expressed in percentage points and, for an arch or dome, $F = \text{rise-to-span ratio multiplied by } 32$.

2.3.14.2 SPECIAL PURPOSE ROOFS.

Roofs that have an occupancy function, such as roof gardens, assembly purposes, or other special purposes are permitted to have their uniformly distributed live load reduced in accordance with the requirements of Section 2.3.13.

Table 2.3.5 Live Load Element Factor, K_{LL}

Element	K_{LL}
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified including: Edge beams with cantilever slabs Cantilever beams One-way slabs Two-way slabs Members without provisions for continuous shear transfer normal to their span	1

^a In lieu of the preceding values, K_{LL} is permitted to be calculated.

2.4 WIND LOADS

2.4.1 GENERAL

Scope: Buildings and other structures, including the Main Wind-Force Resisting System (MWFRS) and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein.

Allowed Procedures: The design wind loads for buildings and other structures, including the MWFRS and component and cladding elements thereof, shall be determined using one of the following procedures:

- (1) Method 1—Simplified Procedure as specified in Section 2.4.4 for buildings and structures meeting the requirements specified therein;
- (2) Method 2—Analytical Procedure as specified in Section 2.4.5 for buildings and structures meeting the requirements specified therein;
- (3) Method 3—Wind Tunnel Procedure as specified in Section 2.4.18.

Wind Pressures: Acting on opposite faces of each building surface. In the calculation of design wind loads for the MWFRS and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

2.4.1.1 MINIMUM DESIGN WIND LOADING

The design wind load, determined by any one of the procedures specified in Section 2.4.1, shall be not less than specified in this section.

Main Wind-Force Resisting System: The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building or other structure shall not be less than 0.5 kN/m^2 multiplied by the area of the

building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than 0.5 kN/m^2 multiplied by the area A_f .

Components and Cladding: The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 0.5 kN/m^2 acting in either direction normal to the surface.

2.4.2 DEFINITIONS

The following definitions apply only to the provisions of Section 2.4:

APPROVED: Acceptable to the authority having jurisdiction.

BASIC WIND SPEED, V : Three-second gust speed at 10 m above the ground in Exposure B (see Section 2.4.8.3) having a return period of 50 years.

BUILDING, ENCLOSED: A building that does not comply with the requirements for open or partially enclosed buildings.

BUILDING ENVELOPE: Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

BUILDING AND OTHER STRUCTURE, FLEXIBLE: Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

BUILDING, LOW-RISE: Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h less than or equal to 18.3 m.
2. Mean roof height h does not exceed least horizontal dimension.

BUILDING, OPEN: A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$ where

A_o = total area of openings in a wall that receives positive external pressure (m^2).

A_g = the gross area of that wall in which A_o is identified (m^2).

BUILDING, PARTIALLY ENCLOSED: A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds 0.37 m^2 or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

$$1. A_o > 1.10A_{oi}$$

$$2. A_o > 0.37 \text{ m}^2 \text{ or } > 0.01A_g, \text{ whichever is smaller,}$$

$$\text{and } A_{oi}/A_{gi} \leq 0.20$$

where

A_o, A_g are as defined for Open Building

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o , in m^2 .

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g , in m^2 .

BUILDING OR OTHER STRUCTURE, REGULAR SHAPED: A building or other structure having no unusual geometrical irregularity in spatial form.

BUILDING OR OTHER STRUCTURES, RIGID: A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.

BUILDING, SIMPLE DIAPHRAGM: A building in which both windward and leeward wind loads are transmitted through floor and roof diaphragms to the same vertical MWFRS (e.g., no structural separations).

COMPONENTS AND CLADDING: Elements of the building envelope that do not qualify as part of the MWFRS.

DESIGN FORCE, F : Equivalent static force to be used in the determination of wind loads for open buildings and other structures.

DESIGNPRESSURE, p : Equivalent static pressure to be used in the determination of wind loads for buildings.

EAVE HEIGHT, h : The distance from the ground surface adjacent to the building to the roof eave line at a particular wall. If the height of the eave varies along the wall, the average height shall be used.

EFFECTIVE WIND AREA, A : The area used to determine GC_p . For component and cladding elements, the effective wind area in Figs. 2.4.11 through 2.4.17 and 2.4.19 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

ESCARPMENT: Also known as scarp, with respect to topographic effects in Section 2.4.9, a cliff or steep slope generally separating two levels or gently sloping areas (see Fig. 2.4.4).

FREE ROOF: Roof (monoslope, pitched, or troughed) in an open building with no enclosing walls underneath the roof surface.

GLAZING: Glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls.

GLAZING, IMPACT RESISTANT: Glazing that has been shown by testing in accordance with ASTM E1886 and ASTM E1996 or other approved test methods to withstand the impact of wind-borne missiles likely to be generated in wind-borne debris regions during design winds.

HILL: With respect to topographic effects in Section 2.4.9, a land surface characterized by strong relief in any horizontal direction (see Fig. 2.4.4).

HURRICANE PRONE REGIONS: Areas vulnerable to hurricanes; in Bangladesh these areas include the Sundarbans, southern parts of Barisal and Patuakhali, Hatia, Bhola, eastern parts of Chittagong and Cox's Bazar

IMPACT RESISTANT COVERING: A covering designed to protect glazing, which has been shown by testing in accordance with ASTM E1886 and ASTM E1996 or other approved test methods to withstand the impact of wind-borne debris missiles likely to be generated in wind-borne debris regions during design winds.

IMPORTANCE FACTOR, I : A factor that accounts for the degree of hazard to human life and damage to property.

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS): An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

MEAN ROOF HEIGHT, h : The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10° , the mean roof height shall be the roof heave height.

OPENINGS: Apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as "open" during design winds as defined by these provisions.

RECOGNIZED LITERATURE: Published research findings and technical papers that are approved.

RIDGE: With respect to topographic effects in Section 2.4.9, an elongated crest of a hill characterized by strong relief in two directions (see Fig. 2.4.4).

WIND-BORNE DEBRIS REGIONS: Areas within hurricane prone regions located:

1. Within 1.6 km of the coastal mean high water line where the basic wind speed is equal to or greater than 180 km/h or
2. In areas where the basic wind speed is equal to or greater than 200 km/h.

2.4.3 SYMBOLS AND NOTATION

The following symbols and notation apply only to the provisions of Section 2.4:

A = effective wind area, in m^2

A_f = area of open buildings and other structures either normal to the wind direction or projected on a plane normal to the wind direction, in m^2 .

A_g = the gross area of that wall in which A_o is identified, in m^2 .

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g , in m^2

A_o = total area of openings in a wall that receives positive external pressure, in m^2 .

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o , in m^2

A_{og} = total area of openings in the building envelope in m^2

A_s = gross area of the solid freestanding wall or solid sign, in m^2

a = width of pressure coefficient zone, in m.

B = horizontal dimension of building measured normal to wind direction, in m.

b = mean hourly wind speed factor in Eq. 2.4.14 from Table 2.4.3

$\hat{b}$ = 3-s gust speed factor from Table 2.4.3

C_f = force coefficient to be used in determination of wind loads for other structures

C_N = net pressure coefficient to be used in determination of wind loads for open buildings

C_p = external pressure coefficient to be used in determination of wind loads for buildings

c = turbulence intensity factor in Eq. 2.4.5 from Table 2.4.3

D = diameter of a circular structure or member in m.

D' = depth of protruding elements such as ribs and spoilers in m.

F = design wind force for other structures, in N.

G = gust effect factor

G_f = gust effect factor for MWFRSs of flexible buildings and other structures

GC_{pn} = combined net pressure coefficient for a parapet

GC_p = product of external pressure coefficient and gust effect factor to be used in determination of wind loads for buildings

GC_{pf} = product of the equivalent external pressure coefficient and gust-effect factor to be used in determination of wind loads for MWFRS of low-rise buildings

GC_{pi} = product of internal pressure coefficient and gust effect factor to be used in determination of wind loads for buildings

g_Q = peak factor for background response in Eqs. 2.4.4 and 2.4.8

g_R = peak factor for resonant response in Eq. 2.4.8

g_v = peak factor for wind response in Eqs. 2.4.4 and 2.4.8

H = height of hill or escarpment in Fig. 2.4.4, in m.

h = mean roof height of a building or height of other structure, except that eave height shall be used for roof angle ϑ of less than or equal to 10° , in m.

h_e = roof eave height at a particular wall, or the average height if the eave varies along the wall

I = importance factor

I_z = intensity of turbulence from Eq. 2.4.5

K_1, K_2, K_3 = multipliers in Fig.2.4.4 to obtain K_{zt}

K_d = wind directionality factor in Table 2.4.5

K_h = velocity pressure exposure coefficient evaluated at height $z = h$

K_z = velocity pressure exposure coefficient evaluated at height z

K_{zt} = topographic factor as defined in Section 2.4.9

L = horizontal dimension of a building measured parallel to the wind direction, in m.

L_h = distance upwind of crest of hill or escarpment in Fig.2.4.4 to where the difference in ground elevation is half the height of hill or escarpment, in m.

L_z = integral length scale of turbulence, in m.

L_r = horizontal dimension of return corner for a solid freestanding wall or solid sign from Fig. 2.4.20, in m.

I = integral length scale factor from Table 2.4.3 in m.

N_1 = reduced frequency from Eq. 2.4.12

n_1 = building natural frequency, Hz

p = design pressure to be used in determination of wind loads for buildings, in N/m^2

p_L = wind pressure acting on leeward face in Fig. 2.4.9, in N/m^2

P_{net} = net design wind pressure from Eq. 2.4.2, in N/m^2

P_{net30} = net design wind pressure for Exposure A at $h = 9.1$ m and $I = 1.0$ from Fig. 2.4.3, in N/m^2 .

P_p = combined net pressure on a parapet from Eq. 2.4.20, in N/m^2 .

P_s = net design wind pressure from Eq. 2.4.1, in N/m^2 .

P_{s30} = simplified design wind pressure for Exposure A at $h = 9.1$ m and $I = 1.0$ from Fig. 2.4.2, in N/m^2 .

P_w = wind pressure acting on windward face in Fig.2.4.9, in N/m^2 .

Q = background response factor from Eq. 2.4.6

q = velocity pressure, in N/m^2 .

q_h = velocity pressure evaluated at height $z = h$, in N/m^2

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q_i = velocity pressure for internal pressure determination, in N/m^2 .

q_p = velocity pressure at top of parapet, in N/m^2 .

q_z = velocity pressure evaluated at height z above ground, in N/m^2 .

R = resonant response factor from Eq. 2.4.10

R_B, R_h, R_L = values from Eq. 2.4.13

R_i = reduction factor from Eq. 2.4.16

R_n = value from Eq. 2.4.11

s = vertical dimension of the solid freestanding wall or solid sign from Fig. 2.4.20, in m.

r = rise-to-span ratio for arched roofs.

V = basic wind speed obtained from Fig. 2.4.1 or Table 2.4.1, in m/s. The basic wind speed corresponds to a 3-s gust speed at 10 m above ground in Exposure Category B having an annual probability of occurrence of 0.02.

V_i = unpartitioned internal volume m^3

$\bar{V}_z$ = mean hourly wind speed at height z , m/s.

W = width of building in Figs. 2.4.12 and 2.4.14A and B and width of span in Figs. 2.4.13 and 2.4.15, in m.

X = distance to center of pressure from windward edge in Fig. 2.4.18, in m.

x = distance upwind or downwind of crest in Fig. 2.4.4, in m.

z = height above ground level, in m.

z = equivalent height of structure, in m.

z_g = nominal height of the atmospheric boundary layer used in this standard. Values appear in Table 2.4.3

$z_{\min}$ = exposure constant from Table 2.4.3

α = 3-s gust-speed power law exponent from Table 2.4.3

$\hat{\alpha}$ = reciprocal of α from Table 2.4.3

$\bar{\alpha}$ = mean hourly wind-speed power law exponent in Eq. 2.4.14 from Table 2.4.3

β = damping ratio, percent critical for buildings or other structures

ϵ = ratio of solid area to gross area for solid freestanding wall, solid sign, open sign, face of a trussed tower, or lattice structure

λ = adjustment factor for building height and exposure from Figs. 2.4.2 and 2.4.3

$\bar{\epsilon}$ = integral length scale power law exponent in Eq. 2.4.7 from Table 2.4.3

η = value used in Eq. 2.4.13 (see Section 2.4.10.2)

ϑ = angle of plane of roof from horizontal, in degrees

v = height-to-width ratio for solid sign

2.4.4 METHOD 1—SIMPLIFIED PROCEDURE

2.4.4.1 SCOPE

A building whose design wind loads are determined in accordance with this section shall meet all the conditions of Sections 2.4.4.2 or 2.4.4.3. If a building qualifies only under 2.4.4.2 for design of its components and cladding, then its MWFRS shall be designed by Method 2 or Method 3.

Limitations on Wind Speeds: Variation of basic wind speeds with direction shall not be permitted unless substantiated by any established analytical method or wind tunnel testing.

2.4.4.2 MAIN WIND-FORCE RESISTING SYSTEMS

For the design of MWFRSs the building must meet all of the following conditions:

1. The building is a simple diaphragm building as defined in Section 2.4.2.
2. The building is a low-rise building as defined in Section 2.4.2.
3. The building is enclosed as defined in Section 2.4.2 and conforms to the wind-borne debris provisions of Section 2.4.11.3.
4. The building is a regular-shaped building or structure as defined in Section 2.4.2.
5. The building is not classified as a flexible building as defined in Section 2.4.2.
6. The building does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
7. The building has an approximately symmetrical cross-section in each direction with either a flat roof or a gable or hip roof with $\vartheta \leq 45^\circ$.
8. The building is exempted from torsional load cases as indicated in Note 5 of Fig. 2.4.10, or the torsional load cases defined in Note 5 do not control the design of any of the MWFRSs of the building.

2.4.4.3 COMPONENTS AND CLADDING

For the design of components and cladding the building must meet all the following conditions:

1. The mean roof height h must be less than or equal to 18.3 m ($h \leq 18.3$ m).
2. The building is enclosed as defined in Section 2.4.2 and conforms to the wind-borne debris provisions of Section 2.4.11.3.
3. The building is a regular-shaped building or structure as defined in Section 2.4.2.
4. The building does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
5. The building has either a flat roof, a gable roof with $\vartheta \leq 45^\circ$, or a hip roof with $\vartheta \leq 27^\circ$.

2.4.4.4 DESIGN PROCEDURE

1. The basic wind speed V shall be determined in accordance with Section 2.4.6. The wind shall be assumed to come from any horizontal direction.
2. An importance factor I shall be determined in accordance with Section 2.4.7.
3. An exposure category shall be determined in accordance with Section 2.4.8.3.

4. A height and exposure adjustment coefficient, λ , shall be determined from Fig.2.4.2.

2.4.4.4.1 Main Wind-Force Resisting System.

Simplified design wind pressures, p_s , for the MWFRSs of low-rise simple diaphragm buildings represent the net pressures (sum of internal and external) to be applied to the horizontal and vertical projections of building surfaces as shown in Fig. 2.4.2. For the horizontal pressures (zones A, B, C, D), p_s is the combination of the windward and leeward net pressures. p_s shall be determined by the following equation:

$$p_s = \lambda K_{zt} / p_{s30} \quad (2.4.1)$$

where

λ = adjustment factor for building height and exposure from Fig. 2.4.2

K_{zt} = topographic factor as defined in Section 2.4.9 evaluated at mean roof height, h

I = importance factor as defined in Section 2.4.7

p_{s30} = simplified design wind pressure for Exposure A, at $h = 9.1\text{m}$, and for $I = 1.0$, from Fig. 2.4.2

Minimum Pressures: The load effects of the design wind pressures from this section shall not be less than the minimum load case from Section 2.4.4.1 assuming the pressures, p_s , for zones A, B, C, and D all equal to $+0.5 \text{ kN/m}^2$, while assuming zones E, F, G, and H all equal to zero kN/m^2 .

2.4.4.4.2 Components and Cladding

Net design wind pressures, p_{net} , for the components and cladding of buildings designed using Method 1 represent the net pressures (sum of internal and external) to be applied normal to each building surface as shown in Fig. 2.4.3.

p_{net} shall be determined by the following equation:

$$p_{net} = \lambda K_{zt} / P_{net30} \quad (2.4.2)$$

where

λ = adjustment factor for building height and exposure from Fig. 2.4.3

K_{zt} = topographic factor as defined in Section 2.4.9 evaluated at mean roof height, h

I = importance factor as defined in Section 2.4.7

p_{net30} = net design wind pressure for Exposure A, at $h = 9.1 \text{ m}$, and for $I = 1.0$, from Fig. 2.4.3

Minimum Pressures : The positive design wind pressures, p_{net} , from this section shall not be less than $+0.5 \text{ kN/m}^2$, and the negative design wind pressures, p_{net} , from this section shall not be less than -0.5 kN/m^2 .

2.4.4.4.3 Air Permeable Cladding

Design wind loads determined from Fig.2.4.3 shall be used for all air permeable cladding unless approved test data or the recognized literature demonstrate lower loads for the type of air permeable cladding being considered.

2.4.5 METHOD 2—ANALYTICAL PROCEDURE

2.4.5.1 SCOPES AND LIMITATIONS

A building or other structure whose design wind loads are determined in accordance with this section shall meet all of the following conditions:

1. The building or other structure is a regular-shaped building or structure as defined in Section 2.4.2.

2. The building or other structure does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

The provisions of this section take into consideration the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings or other structures. Buildings or other structures not meeting the requirements of Section 2.4.4, or having unusual shapes or response characteristics shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure specified in Section 0.

2.4.5.2 SHIELDING.

There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

2.4.5.3 AIR PERMEABLE CLADDING

Design wind loads determined from Section 2.4.5 shall be used for air permeable cladding unless approved test data or recognized literature demonstrate lower loads for the type of air permeable cladding being considered.

2.4.5.4 DESIGN PROCEDURE

1. The basic wind speed V and wind directionality factor K_d shall be determined in accordance with Section 2.4.6.
2. An importance factor I shall be determined in accordance with Section 2.4.7.
3. An exposure category or exposure categories and velocity pressure exposure coefficient K_z or K_h , as applicable, shall be determined for each wind direction in accordance with Section 2.4.8.
4. A topographic factor K_{zt} shall be determined in accordance with Section 2.4.9.
5. A gust effect factor G or G_{fj} , as applicable, shall be determined in accordance with Section 2.4.10.
6. An enclosure classification shall be determined in accordance with Section 2.4.11.
7. Internal pressure coefficient GC_{pi} shall be determined in accordance with Section 2.4.12.1.
8. External pressure coefficients C_p or GC_{pf} , or force coefficients C_f , as applicable, shall be determined in accordance with Section 2.4.12.2 or 2.4.12.3, respectively.
9. Velocity pressure q_z or q_h , as applicable, shall be determined in accordance with Section 2.4.11.5.
10. Design wind load p or F shall be determined in accordance with Sections 2.4.13.

2.4.6 BASIC WIND SPEED

The basic wind speed, V , used in the determination of design wind loads on buildings and other structures shall be as given in Fig. 2.4.1 except as provided in Section 2.4.6.1. The wind shall be assumed to come from any horizontal direction.

2.4.6.1 SPECIAL WIND REGIONS

The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Fig. 2.4.1. Mountainous terrain, gorges, and special regions shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Fig. 2.4.1 to account for higher local wind speeds. Such adjustment shall be based on adequate meteorological information and other necessary data.

2.4.6.2 LIMITATION

Tornadoes have not been considered in developing the basic wind-speed distributions.

2.4.6.3 WIND DIRECTIONALITY FACTOR

The wind directionality factor, K_d , shall be determined from Table 2.4.5. This factor shall only be applied when used in conjunction with load combinations specified in Chapter 2 of Part 6 of this code.

2.4.7 IMPORTANCE FACTOR

An importance factor, I , for the building or other structure shall be determined from Table 2.4.2 based on building and structure categories listed in Section 1.2.4.

2.4.8 EXPOSURE

For each wind direction considered, the upwind exposure category shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities.

2.4.8.1 WIND DIRECTIONS AND SECTORS

For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45° either side of the selected wind direction.

The exposures in these two sectors shall be determined in accordance with Sections 2.4.8.2 and 2.4.8.3 and the exposure resulting in the highest wind loads shall be used to represent the winds from that direction.

2.4.8.2 SURFACE ROUGHNESS CATEGORIES

A ground surface roughness within each 45° sector shall be determined for a distance upwind of the site as defined in Section 2.4.8.3 from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 2.4.8.3.

Surface Roughness A: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness B: Open terrain with scattered obstructions having heights generally less than 9.1 m. This category includes flat open country, grasslands, and all water surfaces in hurricane prone regions.

Surface Roughness C: Flat, unobstructed areas and water surfaces outside hurricane prone regions. This category includes smooth mud flats and salt flats.

2.4.8.3 EXPOSURE CATEGORIES

Exposure A: Exposure A shall apply where the ground surface roughness condition, as defined by Surface Roughness A, prevails in the upwind direction for a distance of at least 792 m or 20 times the height of the building, whichever is greater.

EXCEPTION: For buildings whose mean roof height is less than or equal to 9.1 m, the upwind distance may be reduced to 457 m.

Exposure B: Exposure B shall apply for all cases where Exposures A or C do not apply.

Exposure C: Exposure C shall apply where the ground surface roughness, as defined by Surface Roughness C, prevails in the upwind direction for a distance greater than 1,524 m or 20 times the building height, whichever is greater. Exposure C shall extend into downwind areas of Surface Roughness A or B for a distance of 200 m or 20 times the height of the building, whichever is greater.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

EXCEPTION: An intermediate exposure between the preceding categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognized literature.

2.4.8.4 EXPOSURE CATEGORY FOR MAIN WIND-FORCE RESISTING SYSTEM.

Buildings and Other Structures: For each wind direction considered, wind loads for the design of the MWFRS determined from Fig.2.4.6 shall be based on the exposure categories defined in Section 2.4.8.3.

Low-Rise Buildings: Wind loads for the design of the MWFRSs for low-rise buildings shall be determined using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site where external pressure coefficients GC_{pf} given in Fig. 2.4.10 are used.

2.4.8.5 EXPOSURE CATEGORY FOR COMPONENTS AND CLADDING

Components and cladding design pressures for all buildings and other structures shall be based on the exposure resulting in the highest wind loads for any direction at the site.

2.4.8.6 VELOCITY PRESSURE EXPOSURE COEFFICIENT

Based on the exposure category determined in Section 2.4.8.3, a velocity pressure exposure coefficient K_z or K_h , as applicable, shall be determined from Table 2.4.4. For a site located in a transition zone between exposure categories, that is, near to a change in ground surface roughness, intermediate values of K_z or K_h , between those shown in Table 2.4.4, are permitted, provided that they are determined by a rational analysis method defined in the recognized literature.

2.4.9 TOPOGRAPHIC EFFECTS

2.4.9.1 WIND SPEED-UP OVER HILLS, RIDGES, AND ESCARPMENTS

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

- i. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100H) or 3.22 km, whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.
- ii. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 3.22 km radius in any quadrant by a factor of two or more.
- iii. The structure is located as shown in Fig.2.4.4 in the upper one-half of a hill or ridge or near the crest of an escarpment.
- iv. $H/L_h \geq 0.2$.
- v. H is greater than or equal to 4.5 m for Exposures B and C and 18.3 m for Exposure A.

2.4.9.2 TOPOGRAPHIC FACTOR

The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt} :

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \quad (2.4.3)$$

where K_1 , K_2 , and K_3 are given in Fig. 2.4.4.

If site conditions and locations of structures do not meet all the conditions specified in Section 2.4.9.1 then $K_{zt} = 1.0$.

2.4.10 GUST EFFECT FACTOR

2.4.10.1 RIGID STRUCTURES

For rigid structures as defined in Section 2.4.2, the gust-effect factor shall be taken as 0.85 or calculated by the formula:

$$G = 0.925 \frac{1 + 1.7g_Q I_{\bar{z}} Q}{1 + 1.7g_v I_{\bar{z}}} \quad (2.4.4)$$

$$I_{\bar{z}} = c \left(\frac{10}{\bar{z}} \right)^{1/6} \quad (2.4.5)$$

where $I_{\bar{z}}$ = the intensity of turbulence at height $\bar{z}$ where $\bar{z}$ = the equivalent height of the structure defined as $0.6h$, but not less than $z_{\min}$ for all building heights h . $z_{\min}$ and c are listed for each exposure in Table 2.4.3; g_Q and g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B + h}{L_{\bar{z}}} \right)^{0.63}}} \quad (2.4.6)$$

where B , h are defined in Section 2.4.3; and $L_{\bar{z}}$ = the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = l \left(\frac{\bar{z}}{10} \right)^{\bar{\epsilon}} \quad (2.4.7)$$

in which l and $\bar{\epsilon}$ are constants listed in Table 2.4.3.

2.4.10.2 FLEXIBLE OR DYNAMICALLY SENSITIVE STRUCTURES

For flexible or dynamically sensitive structures as defined in Section 2.4.2 (natural period greater than 1.0 second), the gust-effect factor shall be calculated by

$$G_f = 0.925 \left(\frac{1 + 1.7I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7g_v I_{\bar{z}}} \right) \quad (2.4.8)$$

g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2 \ln(3600n_1)} + \frac{0.577}{\sqrt{2 \ln(3600n_1)}} \quad (2.4.9)$$

R , the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_g (0.53 + 0.47 R_L)} \quad (2.4.10)$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} \quad (2.4.11)$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \quad (2.4.12)$$

$$R_{\ell} = \frac{1}{\eta} - \frac{1}{2\eta^2} (1 - e^{-2\eta}) \text{ for } \eta > 0 \quad (2.4.13a)$$

$$R_{\ell} = 1 \text{ for } \eta = 0 \quad (2.4.13b)$$

where the subscript ℓ in Eq. 2.4.13 shall be taken as h , B , and L , respectively, where h , B , and L are defined in Section 2.4.3.

n_1 = building natural frequency

$R_{\ell} = R_h$ setting $\eta = 4.6n_1 EB / \bar{V}_{\bar{z}}$

$R_{\ell} = R_B$ setting $\eta = 4.6n_1 EB / \bar{V}_{\bar{z}}$

$R_{\ell} = R_L$ setting $\eta = 15.4 n_1 EB / \bar{V}_{\bar{z}}$

θ = damping ratio, percent of critical

$\bar{V}_{\bar{z}}$ = mean hourly wind speed at height $\bar{z}$ determined from Eq. 2.4.14.

$$\bar{V}_{\bar{z}} = \bar{b} \left(\frac{\bar{z}}{10} \right)^{\bar{\alpha}} V \quad (2.4.14)$$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 2.4.3 and V is the basic wind speed in km/h.

2.4.10.3 RATIONAL ANALYSIS

In lieu of the procedure defined in Sections 2.4.10.1 and 2.4.10.2, determination of the gust-effect factor by any rational analysis defined in the recognized literature is permitted.

2.4.10.4 LIMITATIONS

Where combined gust-effect factors and pressure coefficients (GC_p , GC_{pi} , and GC_{pf}) are given in figures and tables, the gust-effect factor shall not be determined separately.

2.4.11 ENCLOSURE CLASSIFICATIONS.

2.4.11.1 GENERAL

For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Section 2.4.2.

2.4.11.2 OPENINGS

A determination shall be made of the amount of openings in the building envelope to determine the enclosure classification as defined in Section 2.4.11.3.

2.4.11.3 WIND-BORNE DEBRIS

Glazing in buildings located in wind-borne debris regions shall be protected with an impact-resistant covering or be impact-resistant glazing according to the requirements specified in ASTM E1886 and ASTM E1996 or other approved test methods and performance criteria. The levels of impact resistance shall be a function of Missile Levels and Wind Zones specified in ASTM E1886 and ASTM E1996.

EXCEPTIONS:

- i. Glazing in Category II, III, or IV buildings located over 18.3 m above the ground and over 9.2 m above aggregate surface roofs located within 458 m of the building shall be permitted to be unprotected.
- ii. Glazing in Category I buildings shall be permitted to be unprotected.

2.4.11.4 MULTIPLE CLASSIFICATIONS

If a building by definition complies with both the “open” and “partially enclosed” definitions, it shall be classified as an “open” building. A building that does not comply with either the “open” or “partially enclosed” definitions shall be classified as an “enclosed” building.

2.4.11.5 VELOCITY PRESSURE

Velocity pressure, q_z , evaluated at height z shall be calculated by the following equation:

$$q_z = 0.000613 K_z K_{zt} K_d V^2 I ; \text{ (kN/m}^2\text{)}, V \text{ in m/s} \quad (2.4.15)$$

where K_d is the wind directionality factor, K_z is the velocity pressure exposure coefficient defined in Section 2.4.8.6, K_{zt} is the topographic factor defined in Section 2.4.9.2, and q_h is the velocity pressure calculated using Eq. 2.4.15 at mean roof height h . The numerical coefficient 0.000613 shall be used except where sufficient climatic data are available to justify the selection of a different value of this factor for a design application.

2.4.12 PRESSURE AND FORCE COEFFICIENTS.

2.4.12.1 INTERNAL PRESSURE COEFFICIENTS

Internal Pressure Coefficient. Internal pressure coefficients, GC_{pi} , shall be determined from Fig. 2.4.5 based on building enclosure classifications determined from Section 2.4.11.

Reduction Factor for Large Volume Buildings, R_i : For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, GC_{pi} , shall be multiplied by the following reduction factor, R_i :

$$R_i = 1.0 \quad \text{or} \quad R_i = 0.5 \left(1 + \frac{1}{\sqrt{1 + \frac{V_i}{6951 A_{og}}}} \right) \leq 1.0 \quad (2.4.16)$$

where

A_{og} = total area of openings in the building envelope (walls and roof, in m^2)

V_i = unpartitioned internal volume, in m^3

2.4.12.2 EXTERNAL PRESSURE COEFFICIENTS.

Main Wind-Force Resisting Systems: External pressure coefficients for MWFRSs C_p are given in Figs. 2.4.6, 2.4.7, and 2.4.8. Combined gust effect factor and external pressure coefficients, GC_{pf} , are given in Fig. 2.4.10 for low-rise buildings. The pressure coefficient values and gust effect factor in Fig. 2.4.10 shall not be separated.

Components and Cladding: Combined gust effect factor and external pressure coefficients for components and cladding GC_p are given in Figs. 2.4.11 through 2.4.17. The pressure coefficient values and gust-effect factor shall not be separated.

2.4.12.3 FORCE COEFFICIENTS

Force coefficients C_f are given in Figs. 2.4.20 through 2.4.23.

2.4.12.4 ROOF OVERHANGS

Main Wind-Force Resisting System: Roof overhangs shall be designed for a positive pressure on the bottom surface of windward roof overhangs corresponding to $C_p = 0.8$ in combination with the pressures determined from using Figs. 2.4.6 and 2.4.10.

Components and Cladding: For all buildings, roof overhangs shall be designed for pressures determined from pressure coefficients given in Figs. 2.4.11B,C,D.

2.4.12.5 PARAPETS

Main Wind-Force Resisting System: The pressure coefficients for the effect of parapets on the MWFRS loads are given in Section 2.4.14.2

Components and Cladding: The pressure coefficients for the design of parapet component and cladding elements are taken from the wall and roof pressure coefficients as specified in Section 2.4.14.3.

2.4.13 DESIGN WIND LOADS ON ENCLOSED AND PARTIALLY ENCLOSED BUILDINGS.

2.4.13.1 GENERAL

Sign Convention: Positive pressure acts toward the surface and negative pressure acts away from the surface.

Critical Load Condition: Values of external and internal pressures shall be combined algebraically to determine the most critical load.

Tributary Areas Greater than 65 m²: Component and cladding elements with tributary areas greater than 65 m² shall be permitted to be designed using the provisions for MWFRSs.

2.4.13.2 MAIN WIND-FORCE RESISTING SYSTEMS

Rigid Buildings of All Heights: Design wind pressures for the MWFRS of buildings of all heights shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \quad (\text{kN/m}^2) \quad (2.4.17)$$

where

$q = q_z$ for windward walls evaluated at height z above the ground

$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings

$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering,

shall be treated as an opening in accordance with Section 2.4.11.3. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$)

G = gust effect factor from Section 2.4.10

C_p = external pressure coefficient from Fig. 2.4.6 or 2.4.8

(GC_{pi}) = internal pressure coefficient from Fig. 2.4.5

q and q_i shall be evaluated using exposure defined in Section 2.4.8.3. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces as defined in Figs. 2.4.6 and 2.4.8.

Low-Rise Building: Alternatively, design wind pressures for the MWFRS of low-rise buildings shall be determined by the following equation:

$$p = q_h [(GC_{pf}) - (GC_{pi})] \text{ (kN/m}^2\text{)} \quad (2.4.18)$$

where

q_h = velocity pressure evaluated at mean roof height h using exposure defined in Section 2.4.8.3

(GC_{pf}) = external pressure coefficient from Fig. 2.4.10

(GC_{pi}) = internal pressure coefficient from Fig. 2.4.5

Flexible Buildings: Design wind pressures for the MWFRS of flexible buildings shall be determined from the following equation:

$$p = q G_f C_p - q_i (GC_{pi}) \text{ (kN/m}^2\text{)} \quad (2.4.19)$$

where q , q_i , C_p , and (GC_{pi}) are as defined in Section 2.4.13.2 and G_f = gust effect factor is defined as in Section 2.4.10.

Parapets: The design wind pressure for the effect of parapets on MWFRSs of rigid, low-rise, or flexible buildings with flat, gable, or hip roofs shall be determined by the following equation:

$$p_p = q_p GC_{pn} \text{ (kN/m}^2\text{)} \quad (2.4.20)$$

where

p_p = combined net pressure on the parapet due to the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet

q_p = velocity pressure evaluated at the top of the parapet

GC_{pn} = combined net pressure coefficient

= +1.5 for windward parapet

= -1.0 for leeward parapet

2.4.13.3 DESIGN WIND LOAD CASES

The MWFRS of buildings of all heights, whose wind loads have been determined under the provisions of Sections 2.4.13.2, shall be designed for the wind load cases as defined in Fig. 2.4.9. The eccentricity e for rigid structures shall be measured from the geometric center of the building face and shall be considered for each principal axis (e_x , e_y). The eccentricity e for flexible structures shall be determined from the following equation and shall be considered for each principal axis (e_x , e_y):

$$e = \frac{e_Q + 1.7I_z \sqrt{(g_Q Q e_Q)^2 + (g_R R e_R)^2}}{1 + 1.7I_z \sqrt{(g_Q Q)^2 + (g_R R)^2}} \quad (2.4.21)$$

where

e_Q = eccentricity e as determined for rigid structures in Fig. 2.4.9

e_R = distance between the elastic shear center and center of mass of each floor

I_z, g_Q, Q, g_R, R shall be as defined in Section 2.4.3

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

EXCEPTION: One-story buildings with h less than or equal to 9.1 m, buildings two stories or less framed with light-frame construction, and buildings two stories or less designed with flexible diaphragms need only be designed for Load Case 1 and Load Case 3 in Fig. 2.4.9.

2.4.13.4 COMPONENTS AND CLADDING.

Low-Rise Buildings and Buildings with $h \leq 18.3$ m: Design wind pressures on component and cladding elements of low-rise buildings and buildings with $h \leq 18.3$ m shall be determined from the following equation:

$$p = q_h [(GC_p) - (GC_{pi})] \text{ (kN/m}^2\text{)} \quad (2.4.22)$$

where

q_h = velocity pressure evaluated at mean roof height h using exposure defined in Section 2.4.8.5

(GC_p) = external pressure coefficients given in Figs. 2.4.11 through 2.4.16

(GC_{pi}) = internal pressure coefficient given in Fig. 2.4.5

Buildings with $h > 18.3$ m: Design wind pressures on components and cladding for all buildings with $h > 18.3$ m shall be determined from the following equation:

$$p = q(GC_p) - q_i(GC_{pi}) \text{ (kN/m}^2\text{)} \quad (2.4.23)$$

where

$q = q_z$ for windward walls calculated at height z above the ground

$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings

$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact-resistant covering, shall be treated as an opening in accordance with Section 2.4.11.3. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$)

(GC_p) = external pressure coefficient from Fig. 2.4.17.

(GC_{pi}) = internal pressure coefficient given in Fig. 2.4.5. q and q_i shall be evaluated using exposure defined in Section 2.4.8.3.

2.4.13.5 ALTERNATIVE DESIGN WIND PRESSURES FOR COMPONENTS AND CLADDING IN BUILDINGS WITH $18.3 \text{ M} < h < 27.4 \text{ M}$

Alternative to the requirements of Section 2.4.13.2, the design of components and cladding for buildings with a mean roof height greater than 18.3 m and less than 27.4 m values from Figs. 2.4.11 through 2.4.17 shall be used only if the height to width ratio is one or less (except as permitted by Note 6 of Fig. 2.4.17) and Eq. 2.4.22 is used.

Parapets: The design wind pressure on the components and cladding elements of parapets shall be designed by the following equation:

$$p = q_p(GC_p - GC_{pi}) \quad (2.4.24)$$

where

q_p = velocity pressure evaluated at the top of the parapet

GC_p = external pressure coefficient from Figs. 2.4.11 through 2.4.17

GC_{pi} = internal pressure coefficient from Fig. 2.4.5, based on the porosity of the parapet envelope.

Two load cases shall be considered. Load Case A shall consist of applying the applicable positive wall pressure from Fig. 2.4.11A or 2.4.17 to the front surface of the parapet while applying the applicable negative edge or corner zone roof pressure from Figs. 2.4.11 through 2.4.17 to the back surface. Load Case B shall consist of applying the applicable positive wall pressure from Fig. 2.4.11A or 2.4.17 to the back of the parapet surface, and applying the applicable negative wall pressure from Fig. 2.4.11A or 2.4.17 to the front surface. Edge and corner zones shall be arranged as shown in Figs. 2.4.11 through 2.4.17. GC_p shall be determined for appropriate roof angle and effective wind area from Figs. 2.4.11 through 2.4.17. If internal pressure is present, both load cases should be evaluated under positive and negative internal pressure.

2.4.14 DESIGN WIND LOADS ON OPEN BUILDINGS WITH MONOSLOPE, PITCHED, OR TROUGHED ROOFS.

2.4.14.1 GENERAL

Sign Convention: Plus and minus signs signify pressure acting toward and away from the top surface of the roof, respectively.

Critical Load Condition: Net pressure coefficients C_N include contributions from top and bottom surfaces. All load cases shown for each roof angle shall be investigated.

2.4.14.2 MAIN WIND-FORCE RESISTING SYSTEMS

The net design pressure for the MWFRSs of monoslope, pitched, or troughed roofs shall be determined by the following equation:

$$p = q_h G C_N \quad (2.4.25)$$

where

q_h = velocity pressure evaluated at mean roof height h using the exposure as defined in Section 2.4.8.3 that results in the highest wind loads for any wind direction at the site

G = gust effect factor from Section 2.4.10

C_N = net pressure coefficient determined from Figs. 2.4.18A through 2.4.18D.

For free roofs with an angle of plane of roof from horizontal θ less than or equal to 5° and containing fascia panels, the fascia panel shall be considered an inverted parapet. The contribution of loads on the fascia to the MWFRS loads shall be determined using Section 2.4.13.5 with q_p equal to q_h .

2.4.14.3 COMPONENT AND CLADDING ELEMENTS

The net design wind pressure for component and cladding elements of monoslope, pitched, and troughed roofs shall be determined by the following equation:

$$p = q_h G C_N \quad (2.4.26)$$

where

q_h = velocity pressure evaluated at mean roof height h using the exposure as defined in Section 2.4.8.3 that results in the highest wind loads for any wind direction at the site

G = gust-effect factor from Section 2.4.10

C_N = net pressure coefficient determined from Figs. 2.4.19A through 2.4.19C.

2.4.15 DESIGN WIND LOADS ON SOLID FREE STANDING WALLS AND SOLID SIGNS

The design wind force for solid freestanding walls and solid signs shall be determined by the following formula:

$$F = q_h G C_f A_s \text{ (kN)} \quad (2.4.27)$$

where

q_h = the velocity pressure evaluated at height h (defined in Fig. 2.4.20) using exposure defined in Section 2.4.8.3

G = gust-effect factor from Section 2.4.10

C_f = net force coefficient from Fig. 2.4.20

A_s = the gross area of the solid freestanding wall or solid sign, in m^2

2.4.16 DESIGN WIND LOADS ON OTHER STRUCTURES

The design wind force for other structures shall be determined by the following equation:

$$F = q_z G C_f A_f \text{ (kN)} \quad (2.4.28)$$

where

q_z = velocity pressure evaluated at height z of the centroid of area A_f using exposure defined in Section 2.4.8.3

G = gust-effect factor from Section 2.4.10

C_f = force coefficients from Figs. 2.4.21 through 2.4.23.

A_f = projected area normal to the wind except where C_f is specified for the actual surface area, m^2

2.4.17 ROOFTOP STRUCTURES AND EQUIPMENT FOR BUILDINGS WITH $H \leq 18.3$ M

The force on rooftop structures and equipment with A_f less than $(0.1Bh)$ located on buildings with $h \leq 18.3$ m shall be determined from Eq. 2.4.28, increased by a factor of 1.9. The factor shall be permitted to be reduced linearly from 1.9 to 1.0 as the value of A_f is increased from $(0.1Bh)$ to (Bh) .

2.4.18 METHOD 3—WIND TUNNEL PROCEDURE

2.4.18.1 SCOPE

Wind tunnel tests shall be used where required by Section 2.4.5.1. Wind tunnel testing shall be permitted in lieu of Methods 1 and 2 for any building or structure.

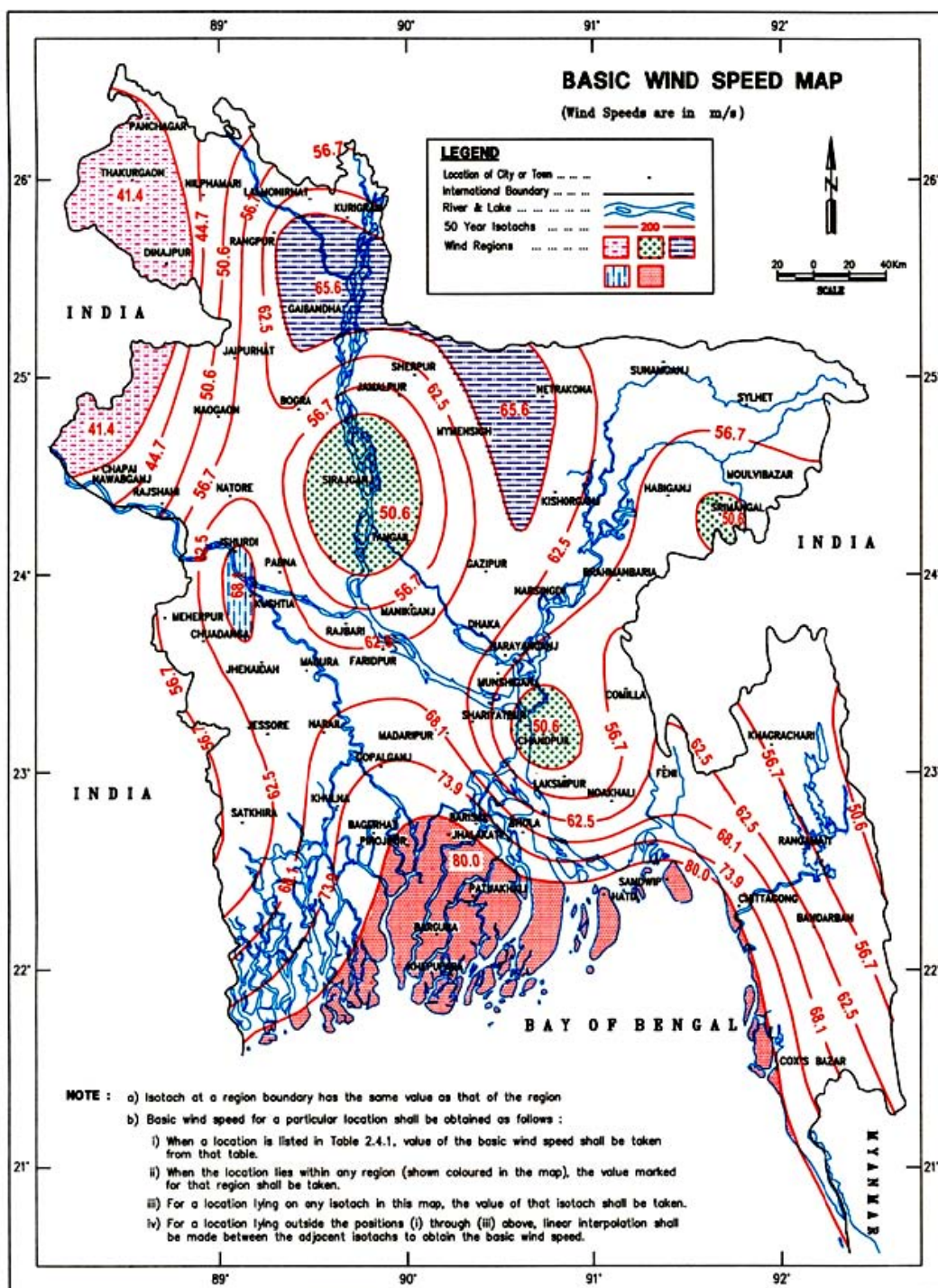
2.4.18.2 TEST CONDITIONS

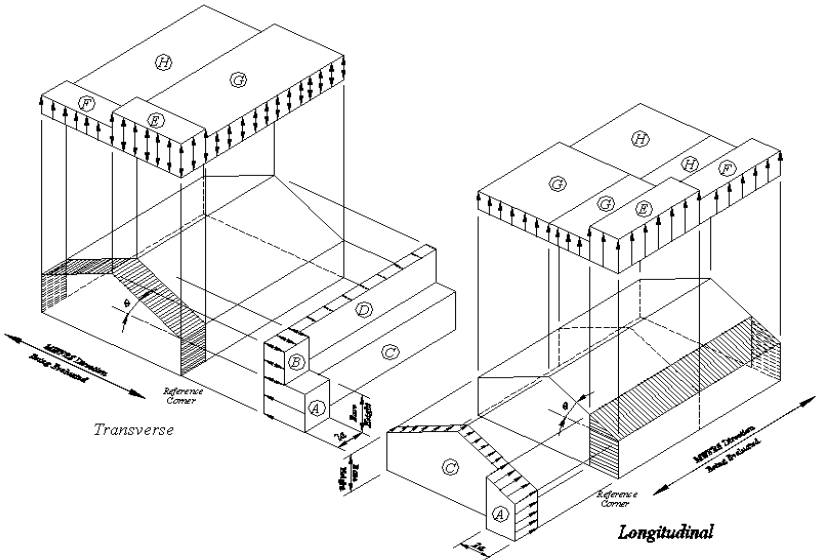
Wind tunnel tests, or similar tests employing fluids other than air, used for the determination of design wind loads for any building or other structure, shall be conducted in accordance with this section. Tests for the determination of mean and fluctuating forces and pressures shall meet all of the following conditions:

- i. The natural atmospheric boundary layer has been modeled to account for the variation of wind speed with height.
- ii. The relevant macro- (integral) length and micro-length scales of the longitudinal component of atmospheric turbulence are modeled to approximately the same scale as that used to model the building or structure.
- iii. The modeled building or other structure and surrounding structures and topography are geometrically similar to their full-scale counterparts, except that, for low-rise buildings meeting the requirements of Section 2.4.5.1, tests shall be permitted for the modeled building in a single exposure site as defined in Section 2.4.8
- iv. The projected area of the modeled building or other structure and surroundings is less than 8 percent of the test section cross-sectional area unless correction is made for blockage.
- v. The longitudinal pressure gradient in the wind tunnel test section is accounted for.
- vi. Reynolds number effects on pressures and forces are minimized.
- vii. Response characteristics of the wind tunnel instrumentation are consistent with the required measurements.

2.4.19 DYNAMIC RESPONSE

Tests for the purpose of determining the dynamic response of a building or other structure shall be in accordance with Section 2.4.18.2. The structural model and associated analysis shall account for mass distribution, stiffness, and damping.

Fig. 2.4.1 Basic wind speed (V_b) map of Bangladesh

Main Wind Force Resisting System – Method 1		$h \leq 18.3 \text{ m}$
Figure 2.4.2	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		
		
Notes: - Pressures shown are applied to the horizontal and vertical projections, for exposure A, at $h=9.1\text{m}$, $I=1.0$, and $K_{zt} = 1.0$. Adjust to other conditions using Equation 2.4.1. - The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 2.4.10) - For the design of the longitudinal MWFRS use $\theta = 0^\circ$, and locate the zone E/F, G/H boundary at the mid-length of the building. - Load cases 1 and 2 must be checked for $25^\circ < \theta \leq 45^\circ$. Load case 2 at 25° is provided only for interpolation between 25° to 30°. - Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively. - For roof slopes other than those shown, linear interpolation is permitted. - The total horizontal load shall not be less than that determined by assuming $p_s = 0$ in zones B & D. - The zone pressures represent the following: Horizontal pressure zones – Sum of the windward and leeward net (sum of internal and external) pressures on vertical projection of: A - End zone of wall C - Interior zone of wall B - End zone of roof D - Interior zone of roof Vertical pressure zones – Net (sum of internal and external) pressures on horizontal projection of: E - End zone of windward roof G - Interior zone of windward roof F - End zone of leeward roof H - Interior zone of leeward roof - Where zone E or G falls on a roof overhang on the windward side of the building, use EOH and GOH for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied. - Notation: a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m. h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles $< 10^\circ$. θ: Angle of plane of roof from horizontal, in degrees.		

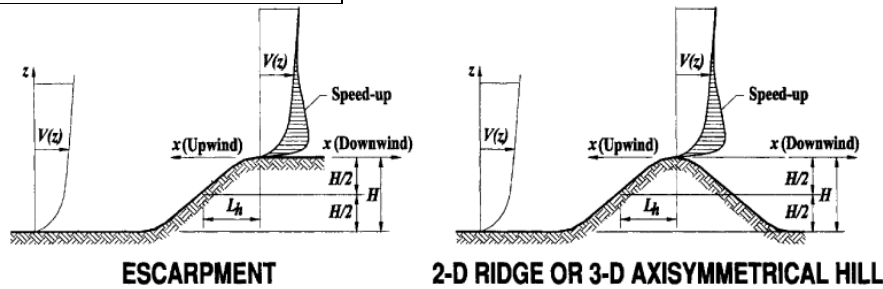
Main Wind Force Resisting System – Method 1		$h \leq 18.3$ m	
Figure 2.4.2 (cont'd)	Design Wind pressures		Walls & Roofs
Enclosed Buildings			
Adjustment Factor			
for Building Height and Exposure, λ			
Mean roof height (m)	Exposure		
	A	B	C
4.6	1.00	1.21	1.47
6	1.00	1.29	1.55
7.6	1.00	1.35	1.61
9.1	1.00	1.40	1.66
10.7	1.05	1.45	1.70
12.2	1.09	1.49	1.74
13.7	1.12	1.53	1.78
15.2	1.16	1.56	1.81
16.8	1.19	1.59	1.84
18.3	1.22	1.62	1.87

Components and Cladding – Method 1		$h \leq 18.3 \text{ m}$
Figure 2.4.3	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		
Flat Roof Hip Roof ($7^\circ < \theta \leq 27^\circ$) Gable Roof ($\theta \leq 7^\circ$) Gable Roof ($7^\circ < \theta \leq 45^\circ$) Interior Zones Roofs - Zone 1/Walls - Zone 4 End Zones Roofs - Zone 2/Walls - Zone 5 Corner Zones Roofs - Zone 3		
Notes: - Pressures shown are applied normal to the surface, for exposure A, at $h = 9.1 \text{ m}$, $l = 1.0$, and $K_{zt} = 1.0$. Adjust to other conditions using Equation 2.4.2. - Plus and minus signs signify pressures acting toward and away from the surfaces, respectively. - For hip roofs with $\theta \leq 25^\circ$, Zone 3 shall be treated as Zone 2. - For effective wind areas between those given, value may be interpolated, otherwise use the value associated with the lower effective wind area. - Notation: a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m. h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles $< 10^\circ$. θ: Angle of plane of roof from horizontal, in degrees.		

Components and Cladding – Method 1									h ≤ 18.3 m	
Figure 2.4.3 (cont'd)				Net Design Wind Pressures					Walls & Roofs	
Enclosed Buildings										
Roof Overhang Net Design Wind Pressure, P _{net30} (kN/m ²) (Exposure A at h = 9.1 m with l= 1.0)										
	Zone	Effective Wind Area (m ²)	Basic Wind Speed V (m/s)							
			40.23	44.7	49.17	53.64	58.11	62.58	67.05	75.99
Roof 0 to 7 degrees	2	0.930	-1.005	-1.239	-1.502	-1.785	-2.096	-2.431	-2.790	-3.584
	2	1.860	-0.986	-1.220	-1.473	-1.756	-2.058	-2.388	-2.742	-3.522
	2	4.648	-0.962	-1.191	-1.440	-1.713	-2.010	-2.330	-2.675	-3.436
	2	9.296	-0.947	-1.168	-1.412	-1.680	-1.971	-2.287	-2.627	-3.373
	3	0.930	-1.656	-2.043	-2.470	-2.943	-3.450	-4.005	-4.594	-5.905
	3	1.860	-1.297	-1.603	-1.938	-2.311	-2.708	-3.144	-3.609	-4.632
	3	4.648	-0.828	-1.024	-1.240	-1.474	-1.727	-2.005	-2.302	-2.957
	3	9.296	-0.479	-0.584	-0.708	-0.842	-0.986	-1.144	-1.311	-1.684
Roof > 7 to 27 degrees	2	0.930	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	2	1.860	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	2	4.648	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	2	9.296	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	3	0.930	-2.187	-2.699	-3.268	-3.885	-4.560	-5.292	-6.072	-7.800
	3	1.860	-1.971	-2.436	-2.948	-3.507	-4.115	-4.775	-5.479	-7.039
	3	4.648	-1.689	-2.086	-2.526	-3.005	-3.526	-4.091	-4.694	-6.034
	3	9.296	-1.479	-1.823	-2.206	-2.627	-3.082	-3.574	-4.106	-5.268
Roof > 27 to 45 degrees	2	0.930	-1.182	-1.460	-1.766	-2.101	-2.464	-2.861	-3.282	-4.216
	2	1.860	-1.148	-1.416	-1.713	-2.038	-2.393	-2.775	-3.182	-4.091
	2	4.648	-1.101	-1.359	-1.641	-1.952	-2.292	-2.660	-3.052	-3.924
	2	9.296	-1.062	-1.311	-1.587	-1.890	-2.220	-2.574	-2.952	-3.795
	3	0.930	-1.182	-1.460	-1.766	-2.101	-2.464	-2.861	-3.283	-4.216
	3	1.860	-1.148	-1.416	-1.713	-2.038	-2.393	-2.775	-3.182	-4.091
	3	4.648	-1.101	-1.359	-1.641	-1.952	-2.292	-2.660	-3.053	-3.923
	3	9.296	-1.062	-1.311	-1.589	-1.890	-2.220	-2.574	-2.952	-3.795
Adjustment Factor										
for Building Height and Exposure, λ										
Mean roof height (m)			Exposure							
			A		B		C			
4.6			1.00		1.21		1.47			
6.1			1.00		1.29		1.55			
7.6			1.00		1.35		1.61			
9.15			1.00		1.40		1.66			
10.7			1.05		1.45		1.70			
12.2			1.09		1.49		1.74			
13.7			1.12		1.53		1.78			
15.2			1.16		1.56		1.81			
16.8			1.19		1.59		1.84			
18.3			1.22		1.62		1.87			
Unit Conversion – 1.0 ft =0.3048 m; 1.0 psf = 0.0929 m ² ; 1.0 psf = 0.0479 KN/m										

Topographic Factor, K_{zt} - Method 2

Figure 2.4.4



Topographic Multipliers for Exposure B

H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

- For values of H/L_h , x/L_h and z/L_h other than those shown, linear interpolation is permitted.
- For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute $2H$ for L_h for evaluating K_2 and K_3 .
- Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
- Notation:

H : Height of hill or escarpment relative to the upwind terrain, in meters.

L_h : Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in meters.

K_1 : Factor to account for shape of topographic feature and maximum speed-up effect.

K_2 : Factor to account for reduction in speed-up with distance upwind or downwind of crest.

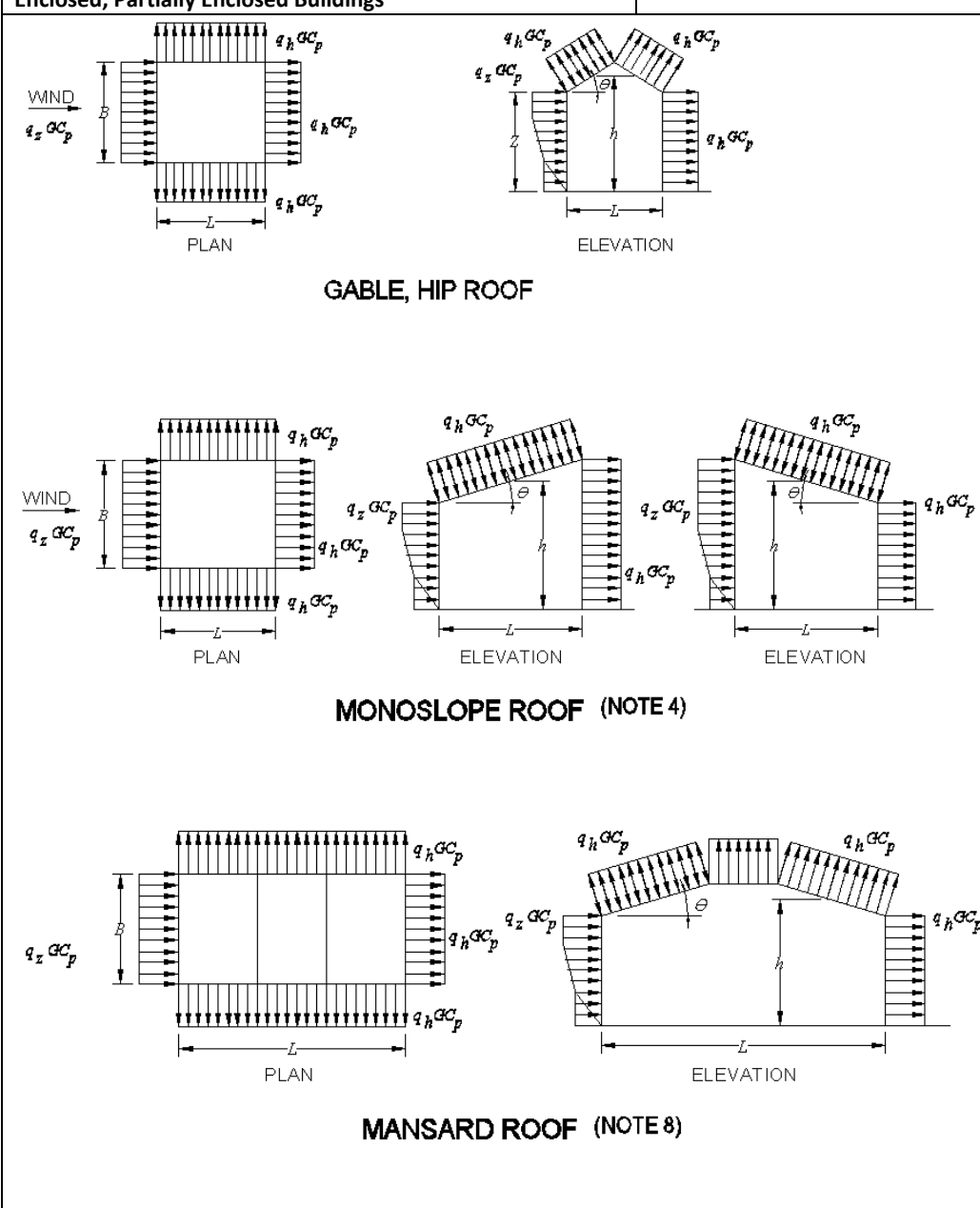
K_3 : Factor to account for reduction in speed-up with height above local terrain.

x : Distance (upwind or downwind) from the crest to the building site, in meters.

z : Height above local ground level, in meters.

W : Horizontal attenuation factor.

γ : Height attenuation factor.

Main Wind Force Resisting System – Method 2		All Heights
Figure 2.4.6	External Pressure Coefficients, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings		
 The figure contains three sets of diagrams, each with a plan view and one or more elevation views, illustrating wind force coefficients for different roof types. Wind is shown coming from the left. GABLE, HIP ROOF: The plan view shows a rectangular building with width B and length L. Windward wall coefficient is $q_z G C_p$, roof coefficient is $q_h G C_p$, and leeward wall coefficient is $q_h G C_p$. The elevation view shows a gable roof with height h and windward wall coefficient $q_h G C_p$. MONOSLOPE ROOF (NOTE 4): The plan view shows a rectangular building with width B and length L. Windward wall coefficient is $q_z G C_p$, roof coefficient is $q_h G C_p$, and leeward wall coefficient is $q_h G C_p$. The elevation view shows a single-slope roof with height h and windward wall coefficient $q_h G C_p$. MANSARD ROOF (NOTE 8): The plan view shows a rectangular building with width B and length L. Windward wall coefficient is $q_z G C_p$, roof coefficient is $q_h G C_p$, and leeward wall coefficient is $q_h G C_p$. The elevation view shows a mansard roof with height h and windward wall coefficient $q_h G C_p$.		

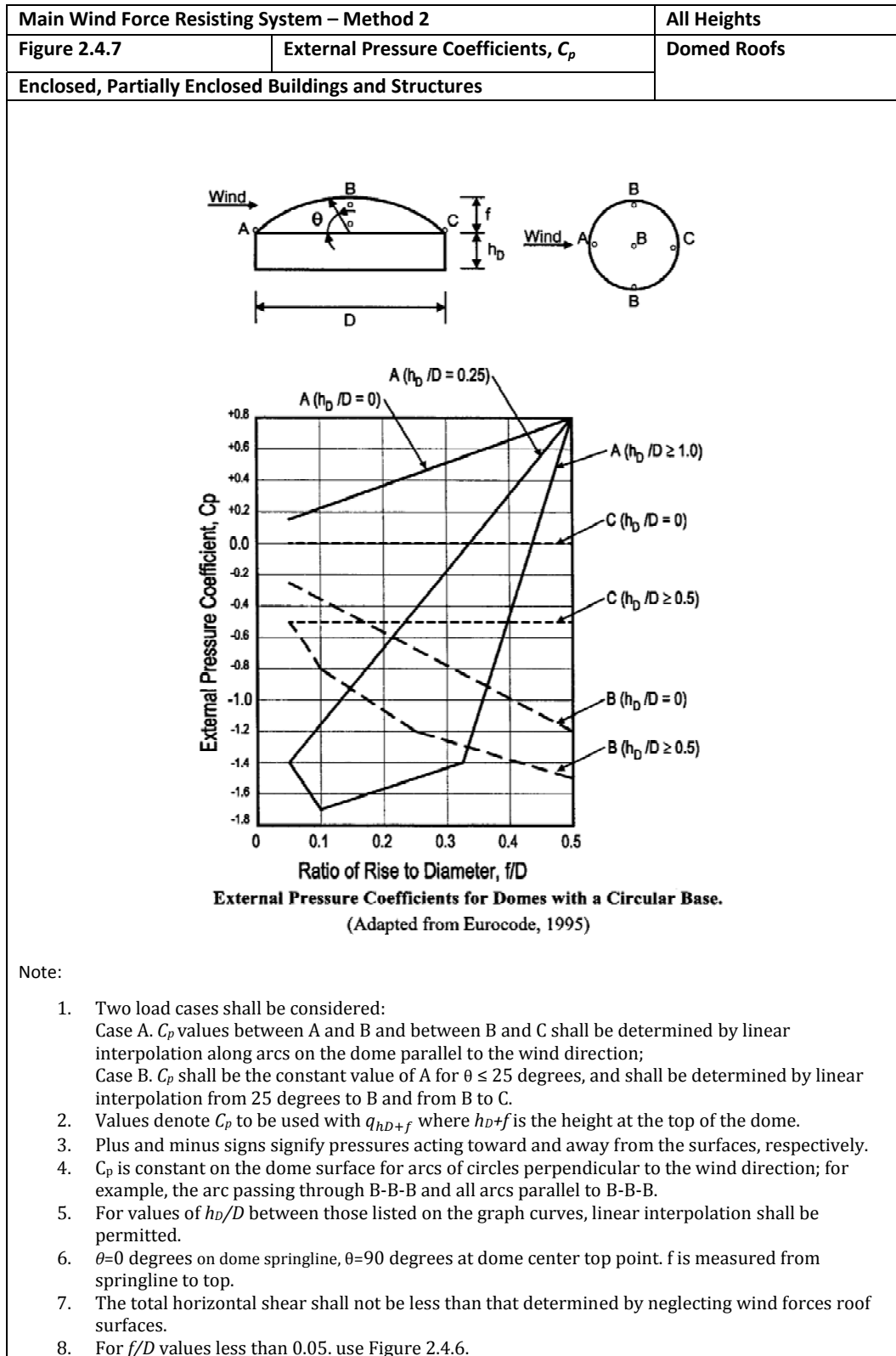
Main Wind Force Resisting System – Method 2										All Heights		
Figure 2.4.6 (con't)			External Pressure Coefficients, C_p							Walls & Roofs		
Enclosed, Partially Enclosed Buildings												
Wall Pressure Coefficients, C_p												
Surface		L/B			C_p			Use With				
Windward Wall		All values			0.8			q_z				
Leeward Wall		0-1			-0.5			q_h				
		2			-0.3							
		≥ 4			-0.2							
Side Wall		All values			-0.7			q_h				

Roof Pressure Coefficients, C_p , for use with q_h												
Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	>60#	10	15	>20
Normal To ridge for $\theta \geq 10^\circ$	≤ 0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.010	-0.3	- 0.5	-0.6
	0.5	-0.9 -0.18	-0.7 - 0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.010	-0.5	- 0.5	-0.6
	≥ 1.0	-1.3** -0.18	-1.0 - 0.18	-0.7 - 0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.010	-0.7	- 0.6	-0.6
Normal To ridge for $\theta < 10^\circ$ and Parallel To ridge for all θ	≤ 0.5	Horizontal distance from Windward edge			C_p		*Value is provided for interpolation purposes ** Value can be reduced linearly with area over which it is applicable as follows					
		0 to $h/2$			-0.9, -0.18							
		$h/2$ to h			-0.9, -0.18							
		H to $2h$			-0.5, -0.18							
		$> 2h$			-0.3, -0.18							
	≥ 1.0	0 to $h/2$			-1.3**, -0.18		Area (m^2)		Reduction Factor			
		$> h/2$			-0.7, -0.18		≤ 9.3 sq m		1.0			
							23.2 sq m		0.9			
					≥ 92.9 sq m		0.8					

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of L/B , h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 2.4.10.
6. Refer to Figure 2.4.7 for domes and Figure 2.4.8 for arched roofs.
7. Notation:
 B : Horizontal dimension of building, in meter, measured normal to wind direction.
 L : Horizontal dimension of building, in meter, measured parallel to wind direction.
 h : Mean roof height in meters, except that eave height shall be used for $\theta < 10$ degrees.
 z : Height above ground, in meters.
 G : Gust effect factor.
 q_z, q_h : Velocity pressure, in N/m^2 , evaluated at respective height.
 θ : Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80° , use $C_p = 0.8$

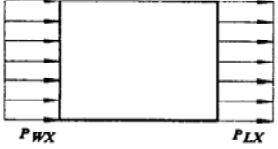
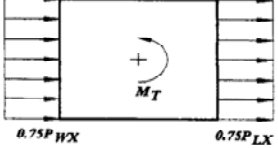
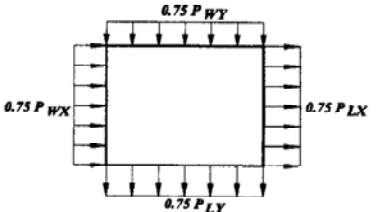
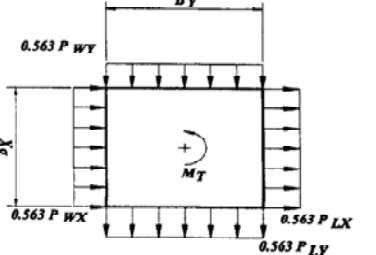


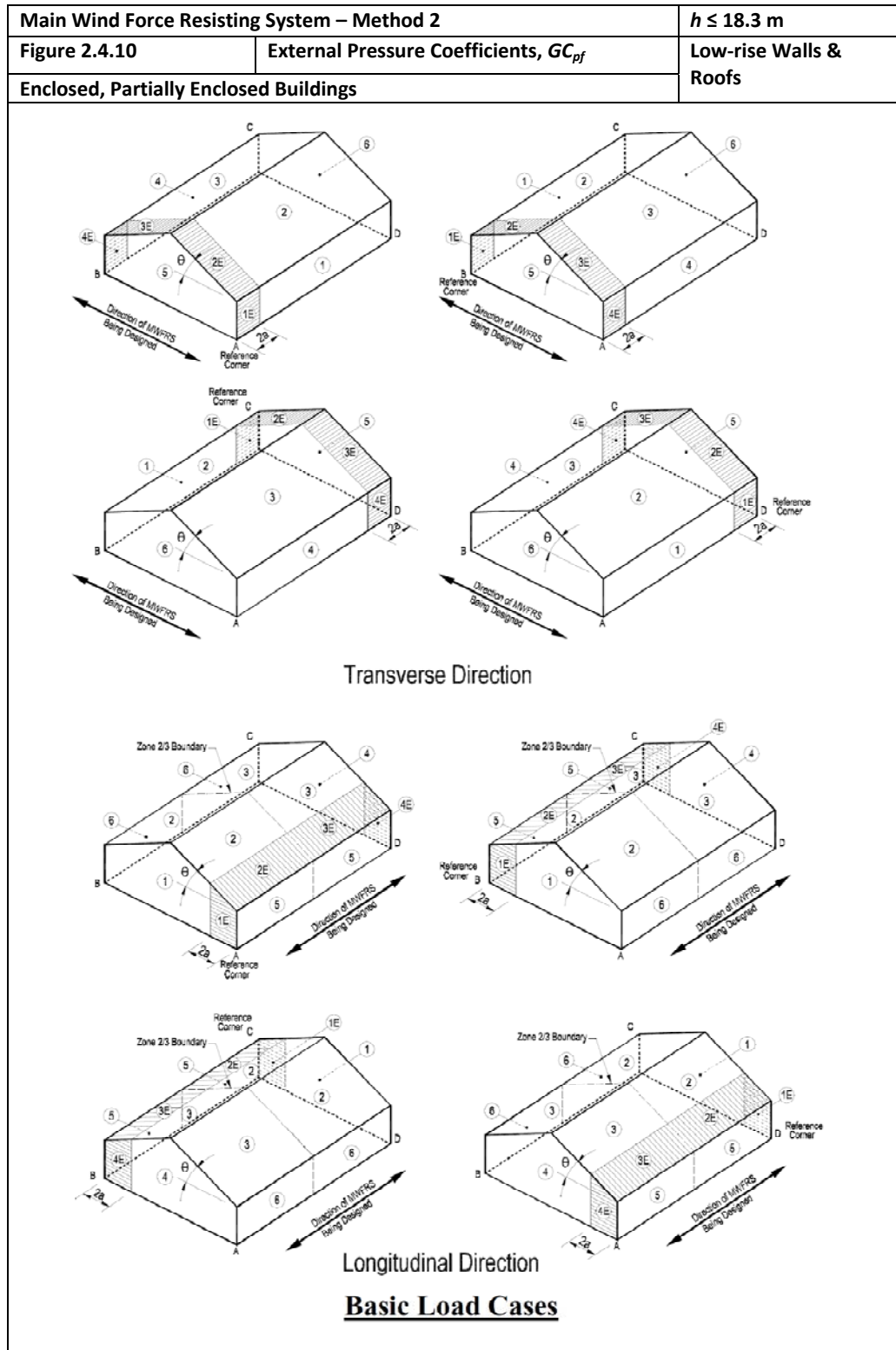
Main Wind Force Res. Sys./ Comp and Clad. - Method 2			All Height	
Figure 2.4.8		External Pressure Coefficients, C_p		Arched Roofs
Enclosed, Partially Enclosed Buildings and Structures				
Condition	Rise-to-span ratio, r	C_p		
		Windward quarter	Center half	Leeward quarter
Roof on elevated structure	$0 < r < 0.2$	-0.9	$-0.7 - r$	-0.5
	$0.2 \leq r < 0.3^*$	$1.5 r - 0.3$	$-0.7 - r$	-0.5
	$0.3 \leq r \leq 0.6$	$2.75 r - 0.7$	$-0.7 - r$	-0.5
Roof springing from ground level	$0 < r \leq 0.6$	$1.4 r$	$-0.7 - r$	-0.5

*When the rise-to-span ratio is $0.2 \leq r \leq 0.3$, alternate coefficients given by $6r - 2.1$ shall also be used for the windward quarter.

Notes:

1. Values listed are for the determination of average load on main wind force resisting systems.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For wind directed parallel to the axis of the arch, use pressure coefficients from Fig. 2.4.6 with wind directed parallel to ridge.
4. For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Fig. 2.4.11 with e based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this table multiplied by 0.87.

Main Wind Force Resisting System – Method 2		All Heights
Figure 2.4.9	Design Wind Load Cases	
 CASE 1  CASE 2  CASE 3  CASE 4 $M_T = 0.75 (P_{WX} + P_{LX}) B_X e_X \quad e_X = \pm 0.15 B_X$ $M_T = 0.75 (P_{WY} + P_{LY}) B_Y e_Y \quad e_Y = \pm 0.15 B_Y$ $M_T = 0.563 (P_{WX} + P_{LX}) B_X e_X + 0.563 (P_{WY} + P_{LY}) B_Y e_Y$ $e_X = \pm 0.15 B_X \quad e_Y = \pm 0.15 B_Y$ Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis. Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis. Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value. Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value. Notes: - Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 2.4.13 as applicable for building of all heights. - Diagrams show plan views of building. - Notation: P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively. P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively. $e(e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively. M_T: Torsional moment per unit height acting about a vertical axis of the building.		



Main Wind Force Resisting System – Method 2							$h \leq 18.3 \text{ m}$				
Figure 2.4.10 (cont'd)		External Pressure Coefficients, GC_{pf}						Low-rise Walls & Roofs			
Enclosed, Partially Enclosed Buildings											
Roof Angle θ (degrees)	Building Surface										
	1	2	3	4	5	6	1E	2E	3E	4E	
	0-5	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
	20	0.53	-0.69	-0.48	-0.43	-0.45	-0.45	0.80	-1.07	-0.69	-0.64
	30-45	0.56	0.21	-0.43	-0.37	-0.45	-0.45	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	-0.45	-0.45	0.69	0.69	-0.48	-0.48	

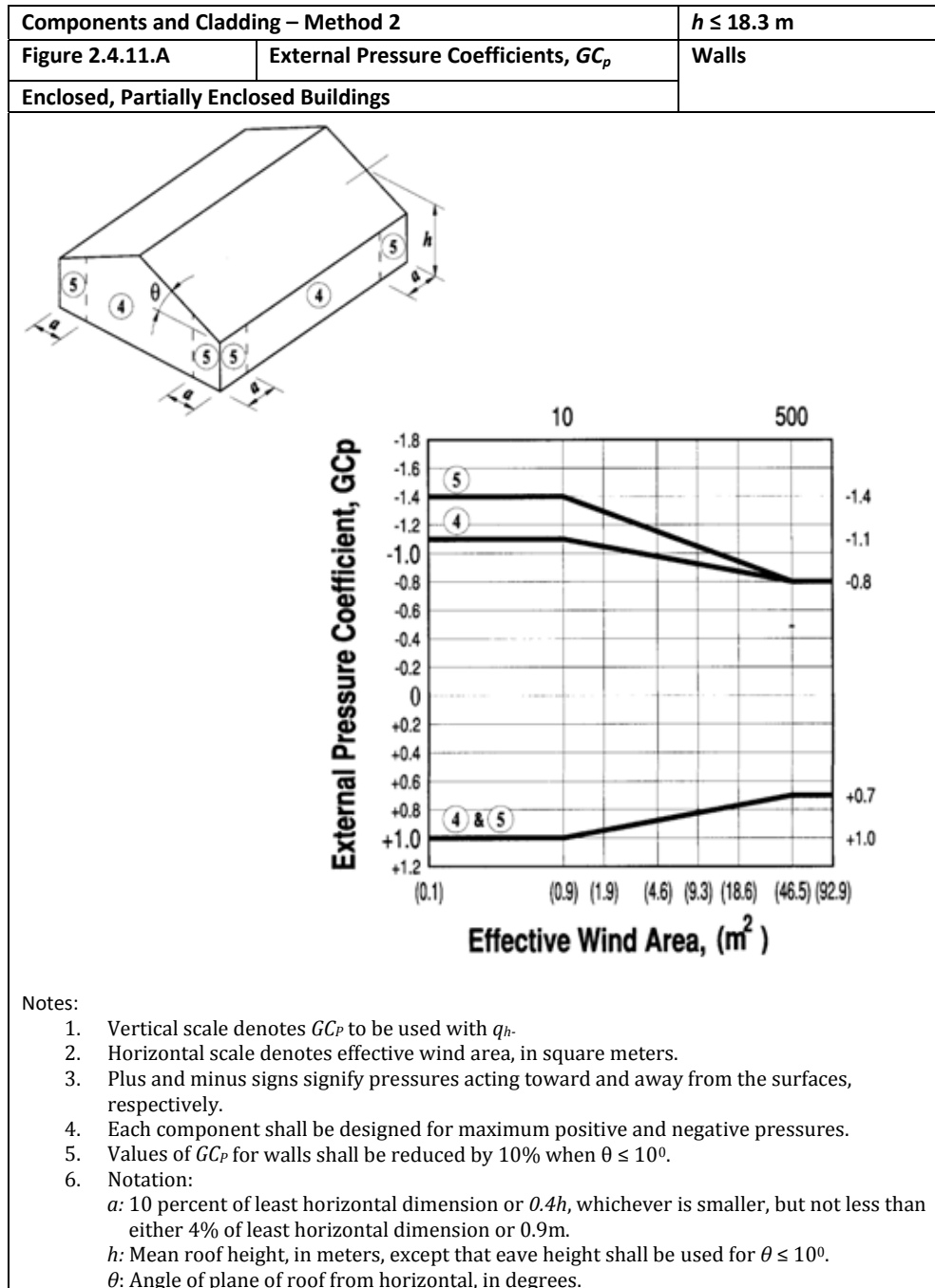
Notes:

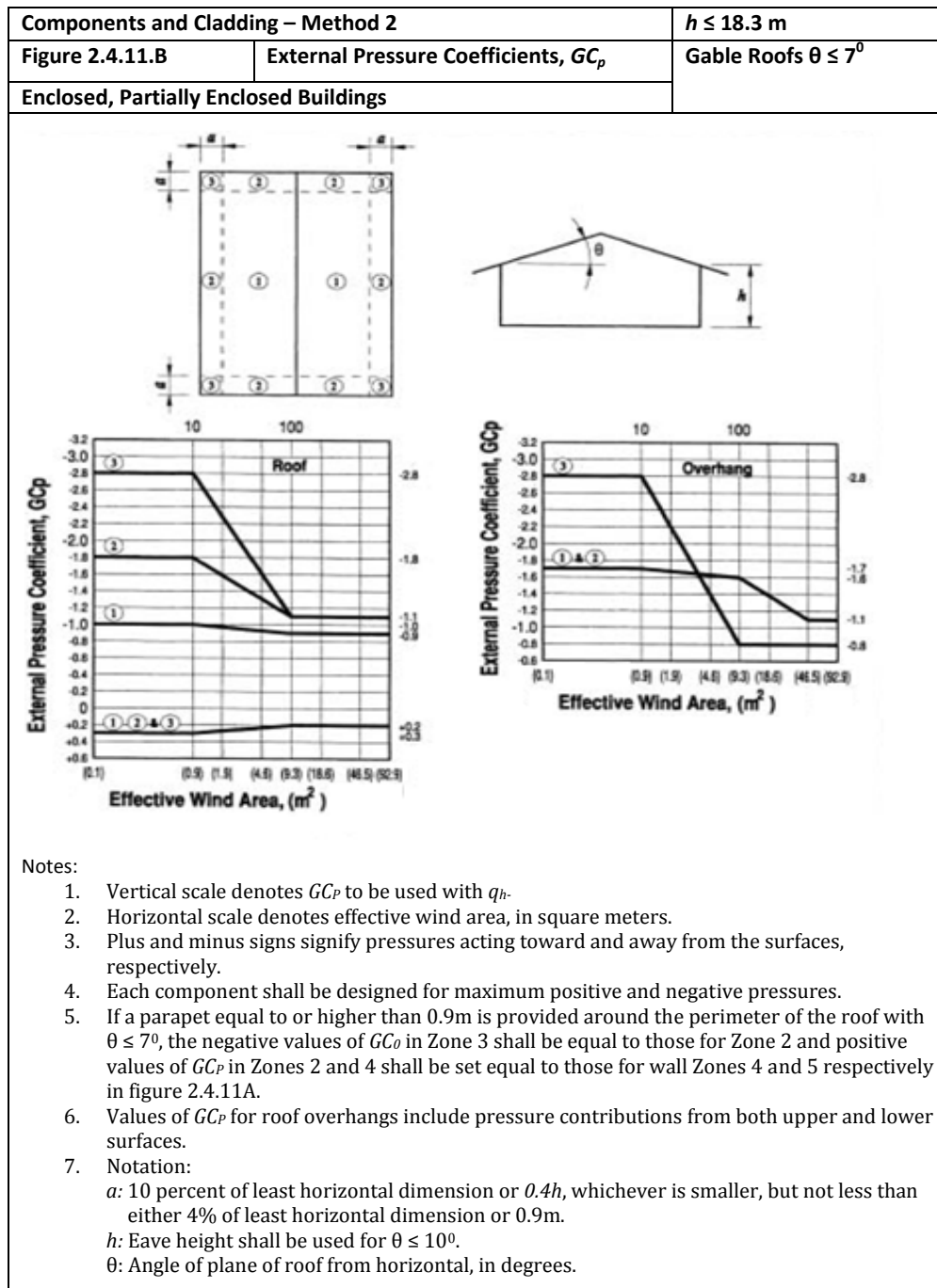
- Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
- For values of θ other than those shown, linear interpolation is permitted.
- The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Reference Corner.
- Combinations of external and internal pressures (see Figure 2.4.5) shall be evaluated as required to obtain the most severe loadings.
- For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T) shall be 25% of the full design wind pressures (zones 1, 2, 3, 4).
Exception: One story buildings with h less than or equal to 9.1m, buildings two stories or less framed with light frame construction, and buildings two stories or less designed with flexible diaphragms need not be designed for the torsional load cases.
Torsional loading shall apply to all eight basic load patterns using the figures below applied at each reference corner.
- Except for moment-resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
- For the design of the MWFRS providing lateral resistance in a direction parallel to a ridge line or for flat roofs, use $\theta = 0^\circ$ and locate the zone 2/3 boundary at the mid-length of the building.
- The roof pressure coefficient GC_{pf} , when negative in Zone 2 or 2E, shall be applied in Zone 2/2E for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5 times the eave height, h_e , at the windward wall, whichever is less; the remainder of Zone 2/2E extending to the ridge line shall use the pressure coefficient GC_{pf} for Zone 3/3E.
- Notation:
 a : 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
 h : Mean roof height, in meters, except that eave height shall be used for $\theta \leq 10^\circ$.
 θ : Angle of plane of roof from horizontal, in degrees.

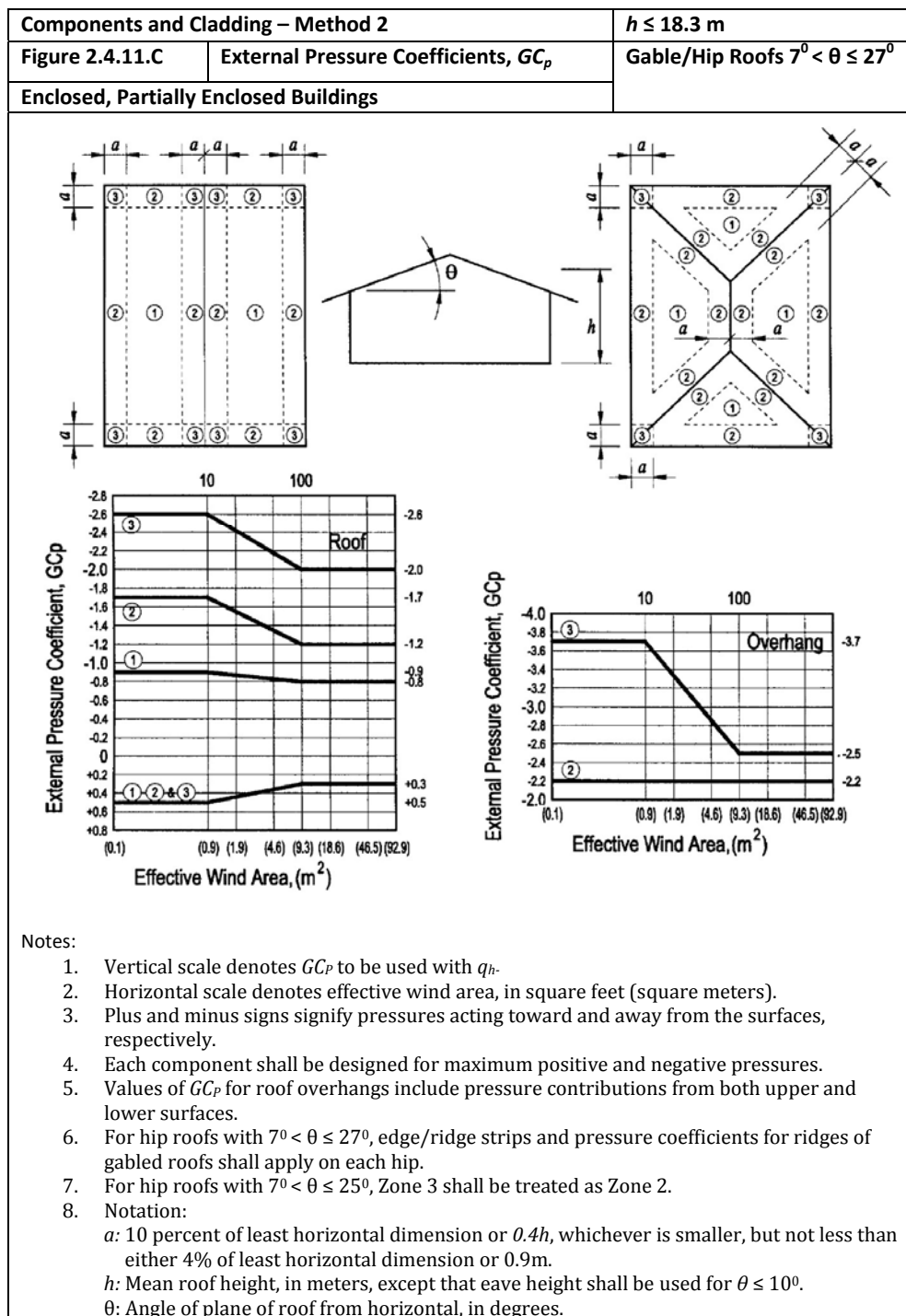
Transverse Direction

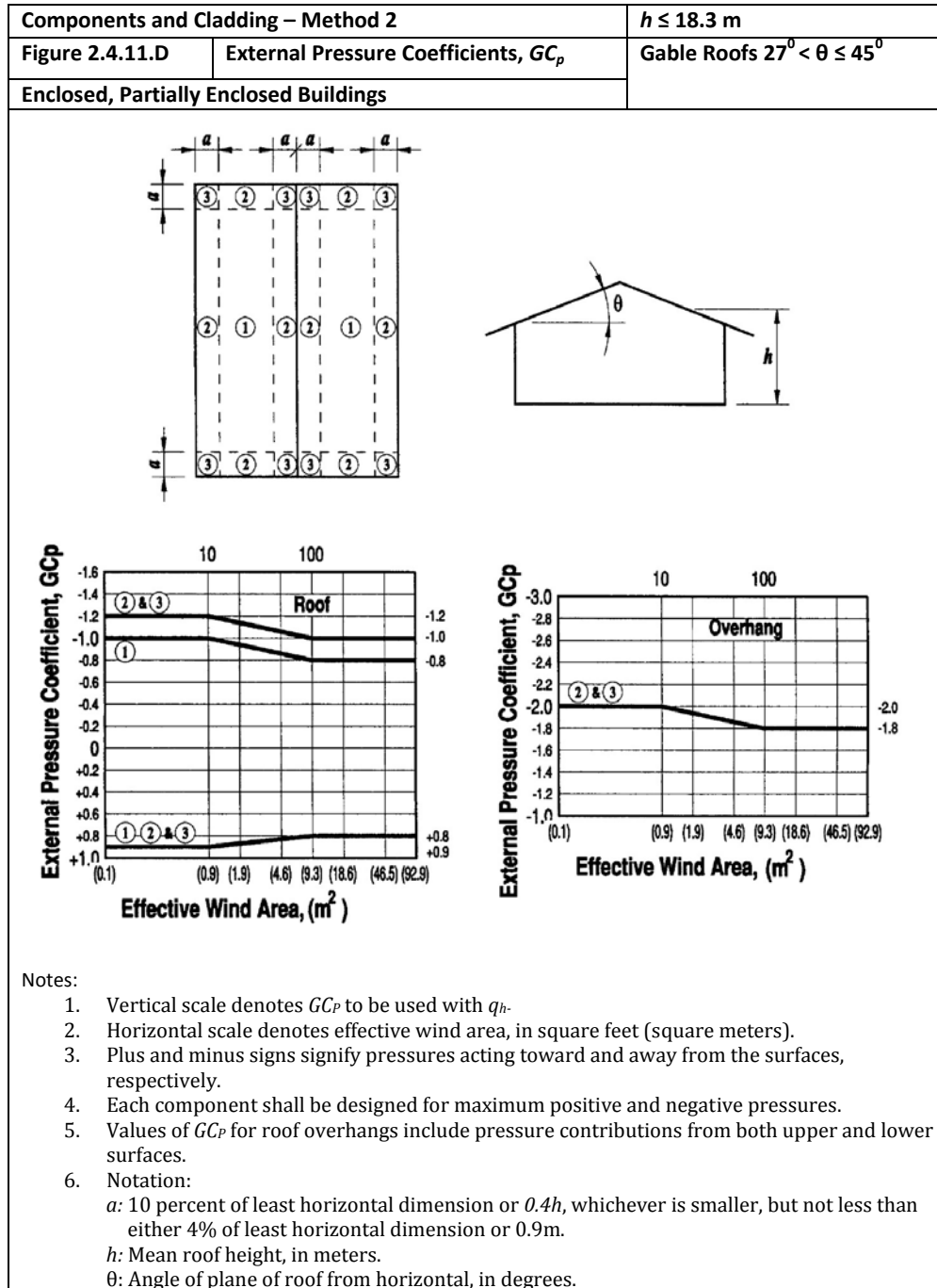
Longitudinal Direction

Torsional Load Cases









Components and Cladding – Method 2		$h \leq 18.3 \text{ m}$
Figure 2.4.12	External Pressure Coefficients, GC_p	Stepped Roofs
Enclosed, Partially Enclosed Buildings		

$h_1 \geq 3 \text{ m}$
 $b = 1.5 h_1$
 $b < 30.5 \text{ m}$
 $\frac{h_i}{h} = 0.3 \text{ to } 0.7$
 $\frac{W_i}{W} = 0.25 \text{ to } 0.75$

Notes:

On the lower level of flat, stepped roofs shown in Fig. 2.4.12, the zone designations and pressure coefficients shown in Fig. 2.4.11B shall apply, except that at the roof-upper wall intersection(s), Zone 3 shall be treated as Zone 2 and Zone 2 shall be treated as Zone 1. Positive values of GC_p equal to those for walls in Fig. 2.4.11A shall apply on the cross-hatched areas shown in Fig. 2.4.12.

Notation:

b : $1.5h_1$ in Fig. 2.4.12, but not greater than 30.5 m.

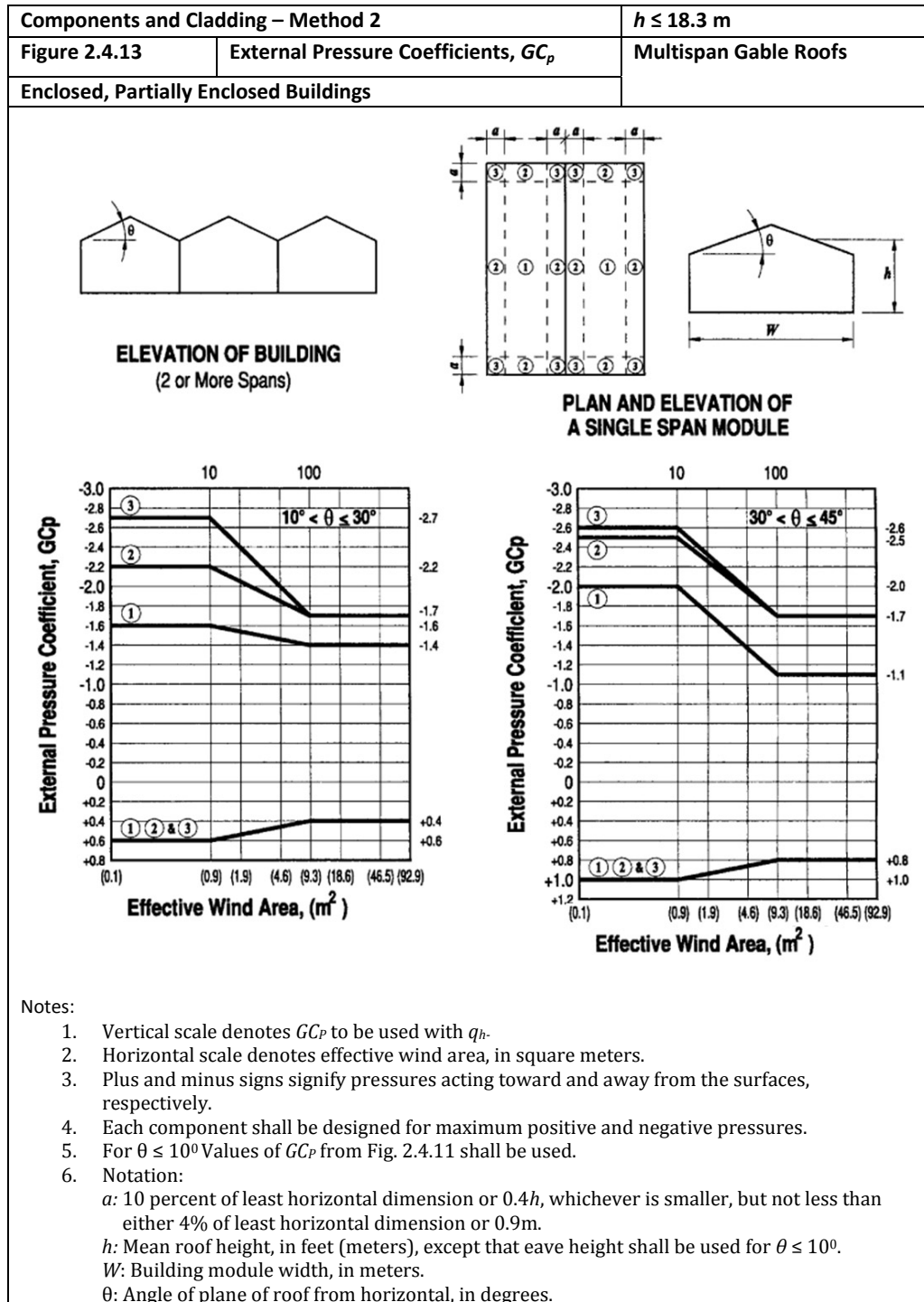
h : Mean roof height, in meters.

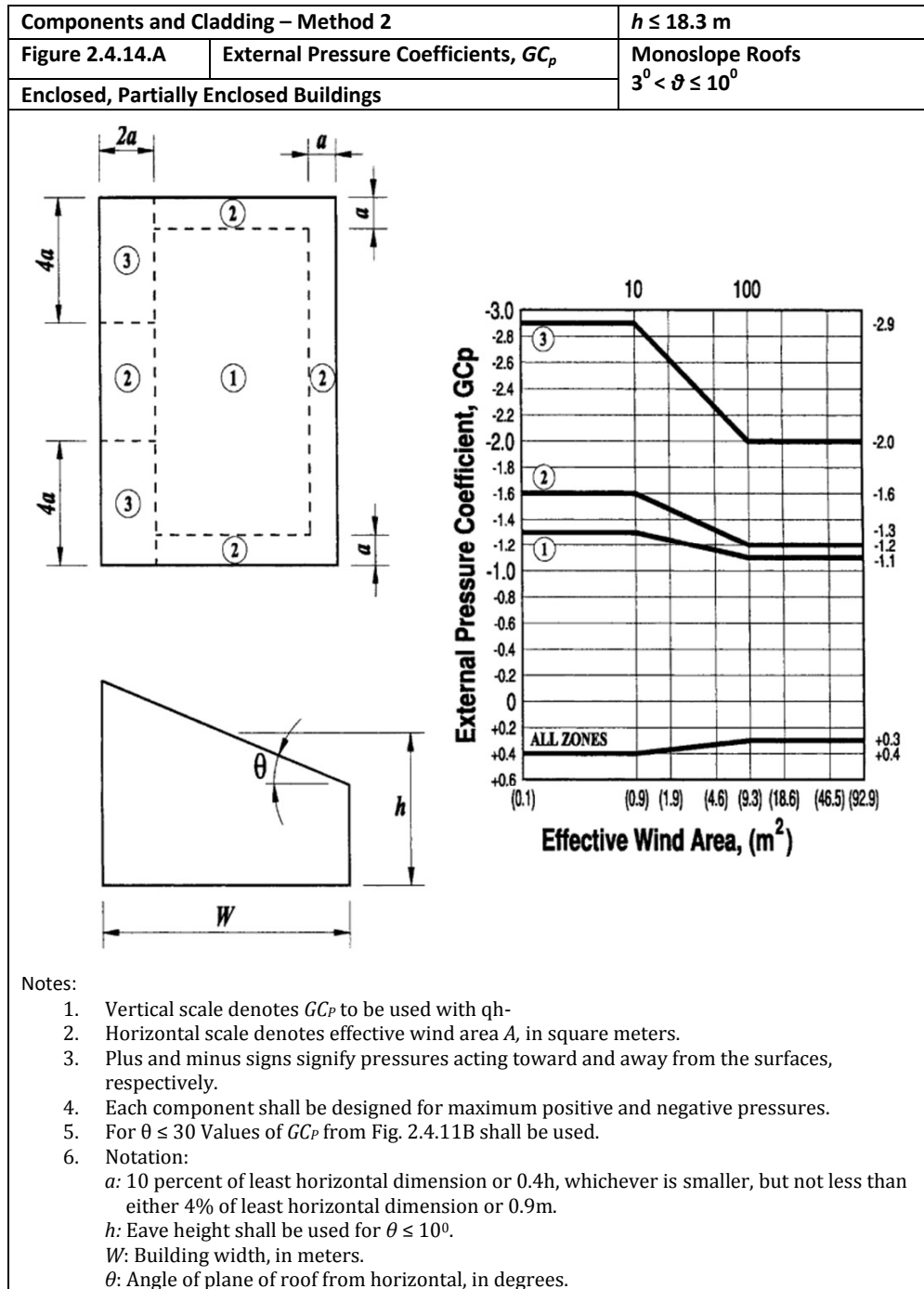
h_i : h_1 or h_2 in Fig. 2.4.12; $h = h_1 + h_2$; $h_1 \geq 3.1 \text{ m}$; $h_i/h = 0.3 \text{ to } 0.7$.

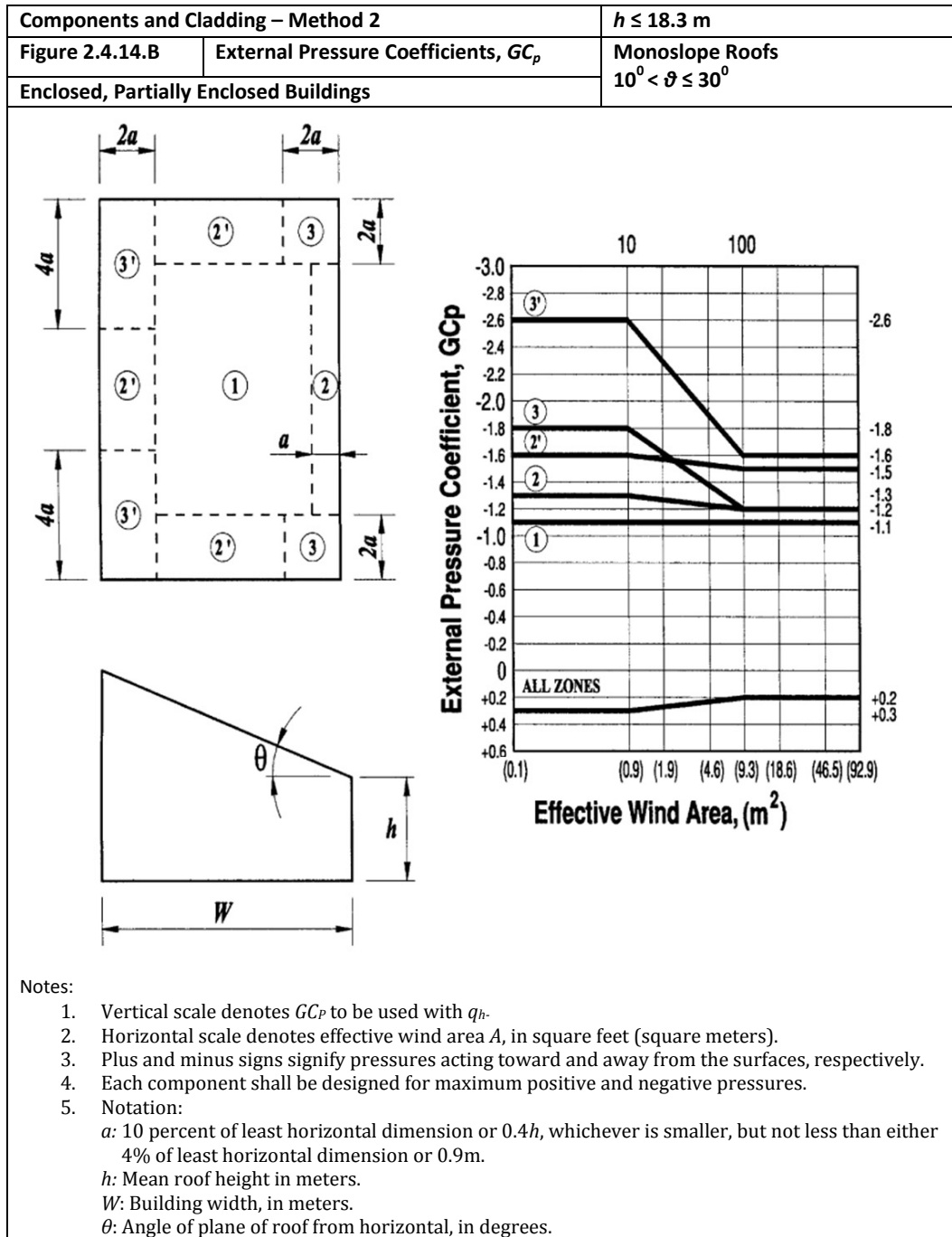
W : Building width in Fig. 2.4.12.

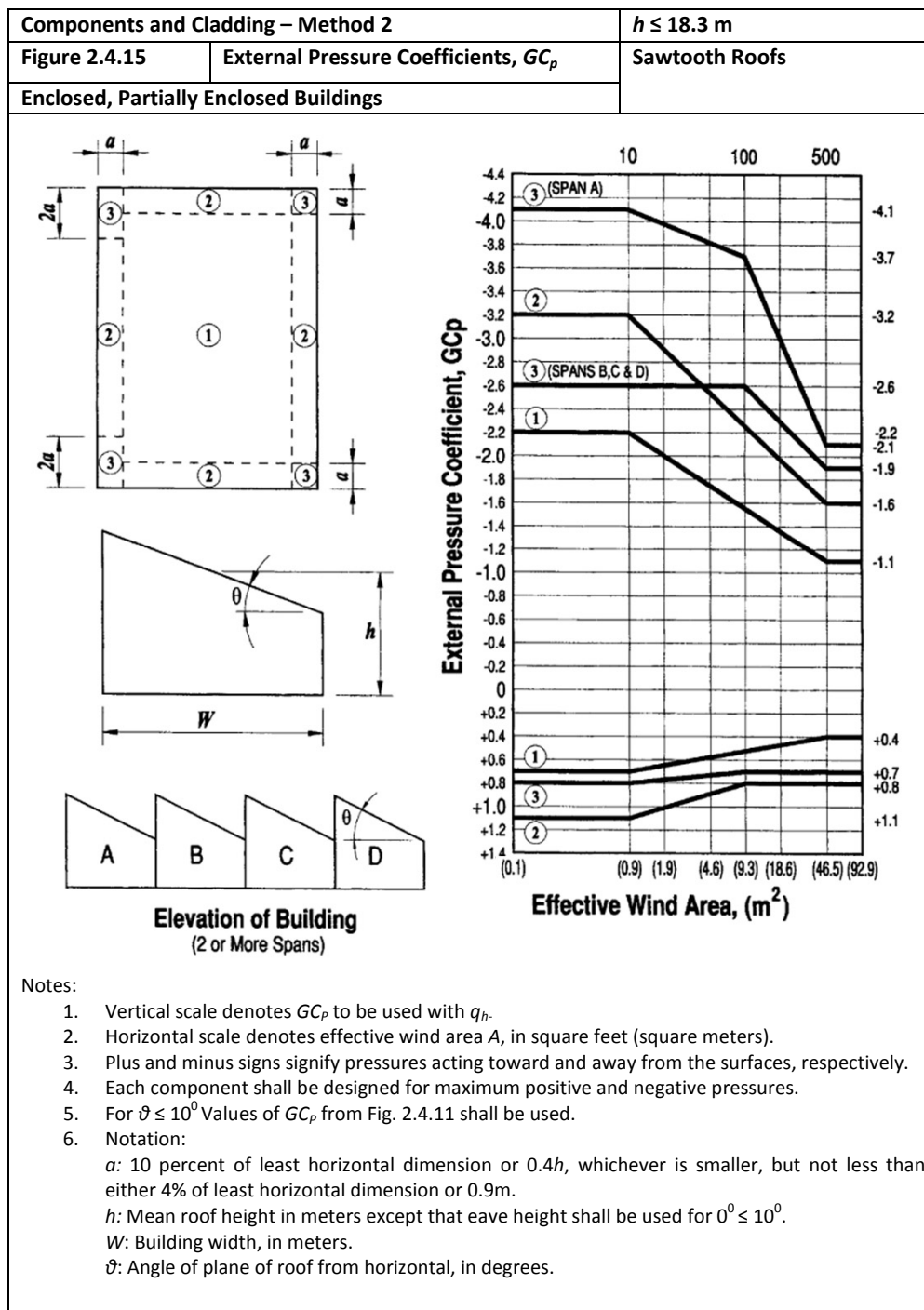
W_i : W_1 or W_2 or W_3 in Fig. 2.4.12. $W = W_1 + W_2$ or $W_1 + W_2 + W_3$; $W_i/W = 0.25 \text{ to } 0.75$.

e : Angle of plane of roof from horizontal, in degrees.

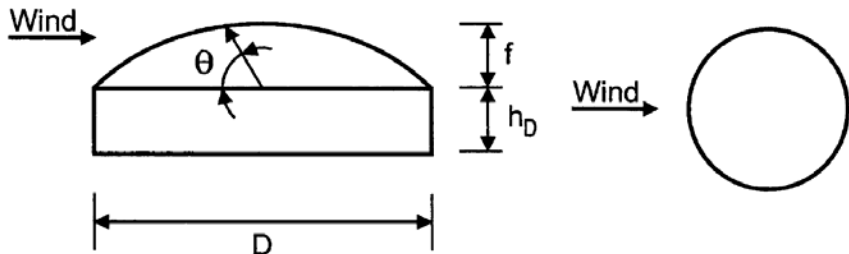








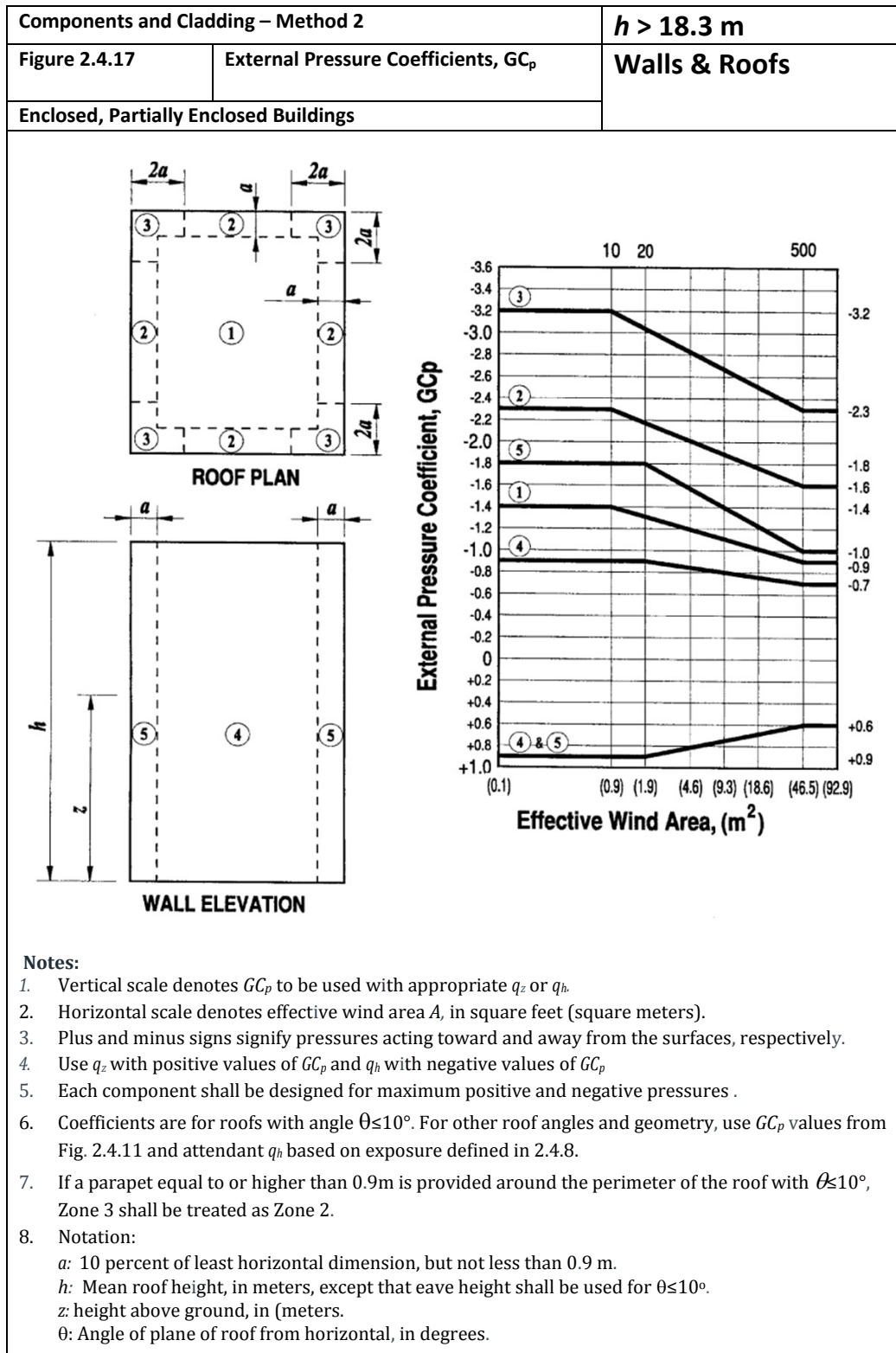
Components and Cladding – Method 2		All Heights
Figure 2.4.16	External Pressure Coefficients, GC_p	Domed Roofs
Enclosed, Partially Enclosed Buildings		



External Pressure Coefficients for Domes with a circular Base			
θ , degrees	Negative Pressures	Positive Pressures	Positive Pressures
	0 – 90	0 – 60	61 – 90
GC_p	-0.9	+0.9	+0.5

Notes:

1. Values denote C_p to be used with $q_{(h_D+f)}$ where h_D+f is the height at the top of the dome.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. Each component shall be designed for maximum positive and negative pressures.
4. Values apply to $0 \leq h_D/D \leq 0.5$, $0.2 \leq f/D \leq 0.5$.
5. $\theta=0$ degrees on dome springline, $\theta=90$ degrees at dome center top point. f is measured from springline to top.



Main Wind Force Resisting System				0.25 ≤ h/L ≤ 1.0	
Figure 2.4.18A		Net Pressure Coefficient, C _N		Monoslope Free Roofs	
Open Buildings				q ≤ 45°, γ= 0°, 180°	

Wind Direction
γ = 0°

Wind Direction
γ = 180°

Roof Angle θ	Load Case	Wind Direction, γ = 0°				Wind Direction, γ=180°			
		Clear Wind Flow		Obstructed Wind Flow		Clear Wind Flow		Obstructed Wind Flow	
		C _{NW}	C _{NL}	C _{NW}	C _{NL}	C _{NW}	C _{NL}	C _{NW}	C _{NL}
0°	A	1.2	0.3	-0.5	-1.2	1.2	0.3	-0.5	-1.2
	B	-1.1	-0.1	-1.1	-0.6	-1.1	-0.1	-1.1	-0.6
7.5°	A	-0.6	-1	-1	-1.5	0.9	1.5	-0.2	-1.2
	B	-1.4	0	-1.7	-0.8	1.6	0.3	0.8	-0.3
15°	A	-0.3	-1.3	-1.1	-1.5	1.3	1.6	0.4	-1.1
	B	-1.9	0	-2.1	-0.6	1.8	0.6	1.2	-0.3
22.5°	A	-1.5	-1.6	-1.5	-1.7	1.7	1.8	0.5	-1
	B	-2.4	-0.3	-2.3	-0.9	2.2	0.7	1.3	0
30°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.1	0.6	-1
	B	-2.5	-0.6	-2.3	-1.1	2.6	1	1.6	0.1
37.5°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.2	0.7	-0.9
	B	-2.4	-0.6	-2.2	-1.1	2.7	1.1	1.9	0.3
45°	A	-1.6	-1.8	-1.3	-1.8	2.2	2.5	0.8	-0.9
	B	-2.3	-0.7	-1.9	-1.2	2.6	1.4	2.1	0.4

Notes:

C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.

Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%.

Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).

For values of e between 7.5° and 45°, linear interpolation is permitted. For values of e less than 7.5°, use Monoslope roof load coefficients.

Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.

All load cases shown for each roof angle shall be investigated.

Notation:

L

:

horizontal dimension of roof, measured in the along wind direction, m

h

:

mean roof height, m

γ

:

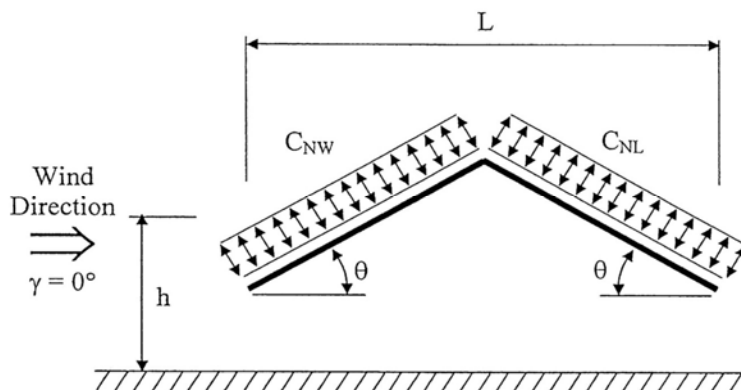
direction of wind, degrees

θ

:

angle of plane of roof from horizontal, degrees

Main Wind Force Resisting System		$0.25 \leq h/L \leq 1.0$
Figure 2.4.18B	Net Pressure Coefficients, C_N	Pitched Free Roofs $\theta \leq 45^\circ, \gamma = 0^\circ, 180^\circ$
Open Buildings		



Roof Angle, θ	Load Case	Wind Direction, $\gamma=0^\circ, 180^\circ$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	1.1	-0.3	-1.6	-1
	B	0.2	-1.2	-0.9	-1.7
15°	A	1.1	-0.4	-1.2	-1
	B	0.1	-1.1	-0.6	-1.6
22.5°	A	1.1	0.1	-1.2	-1.2
	B	-0.1	-0.8	-0.8	-1.7
30°	A	1.3	0.3	-0.7	-0.7
	B	-0.1	-0.9	-0.2	-1.1
37.5°	A	1.3	0.6	-0.6	-0.6
	B	-0.2	-0.6	-0.3	-0.9
45°	A	1.1	0.9	-0.5	-0.5
	B	-0.3	-0.5	-0.3	-0.7

Notes:

C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.

Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%.

Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).

For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.

Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.

All load cases shown for each roof angle shall be investigated.

Notation:

L	: horizontal dimension of roof, measured in the along wind direction, m
h	: mean roof height, m
γ	: direction of wind, degrees
θ	: angle of plane of roof from horizontal, degrees

Main Wind Force Resisting System		$0.25 \leq h/L \leq 1.0$	
Figure 2.4.18C	Net Pressure Coefficients, C_N	Troughed Free Roofs	
Open Buildings		$\theta \leq 45^\circ, \gamma = 0^\circ, 180^\circ$	

Roof Angle, θ	Load Case	Wind Direction, $\gamma=0^\circ, 180^\circ$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	-1.1	0.3	-1.6	-0.5
	B	-0.2	1.2	-0.9	-0.8
15°	A	-1.1	0.4	-1.2	-0.5
	B	0.1	1.1	-0.6	-0.8
22.5°	A	-1.1	-0.1	-1.2	-0.6
	B	-0.1	0.8	-0.8	-0.8
30°	A	-1.3	-0.3	-1.4	-0.4
	B	-0.1	0.9	-0.2	-0.5
37.5°	A	-1.3	-0.6	-1.4	-0.3
	B	0.2	0.6	-0.3	-0.4
45°	A	-1.1	-0.9	-1.2	-0.3
	B	0.3	0.5	-0.3	-0.4

Notes:

C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.

Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%.

Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).

For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.

Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.

All load cases shown for each roof angle shall be investigated.

Notation:

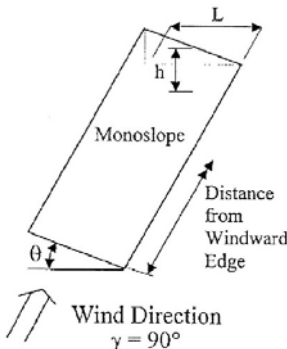
L : horizontal dimension of roof, measured in the along wind direction, m

h : mean roof height, m

γ : direction of wind, degrees

θ : angle of plane of roof from horizontal, degrees

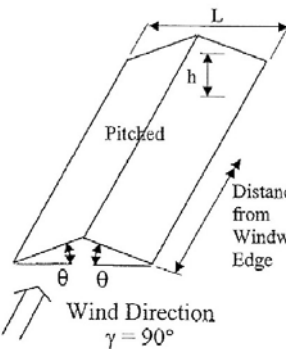
Main Wind Force Resisting System		$0.25 \leq h/L \leq 1.0$
Figure 2.4.18D	Net Pressure Coefficients, C_N	Troughed Free Roofs $\theta \leq 45^\circ, \lambda = 0^\circ, 180^\circ$
Open Buildings		



Monoslope

Distance from Windward Edge

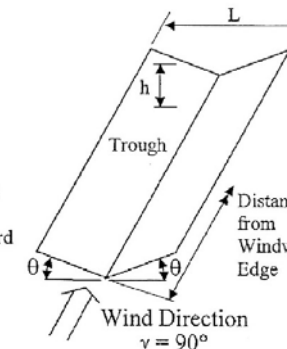
Wind Direction $\gamma = 90^\circ$



Pitched

Distance from Windward Edge

Wind Direction $\gamma = 90^\circ$



Trough

Distance from Windward Edge

Wind Direction $\gamma = 90^\circ$

Horizontal Distance from Windward Edge	Roof Angle θ	Load Case	Clear Wind Flow	Obstructed Wind Flow
			C_N	C_N
$\leq h$	All Shapes	A	-0.8	-1.2
	$\theta \leq 45^\circ$	B	0.8	0.5
$> h, \leq 2h$	All Shapes	A	-0.6	-0.9
	$\theta \leq 45^\circ$	B	0.5	0.5
$> 2h$	All Shapes	A	-0.3	-0.6
	$\theta \leq 45^\circ$	B	0.3	0.3

Notes:

C_N denotes net pressures (contributions from top and bottom surfaces).

Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%.

Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).

Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.

All load cases shown for each roof angle shall be investigated.

For monoslope roofs with theta less than 5 degrees, C_N values shown apply also for cases where gamma = 0 degrees and 0.05 less than or equal to h/L less than or equal to 0.25. See Figure 2.4.18A for other h/L values.

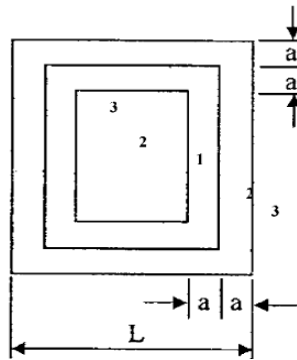
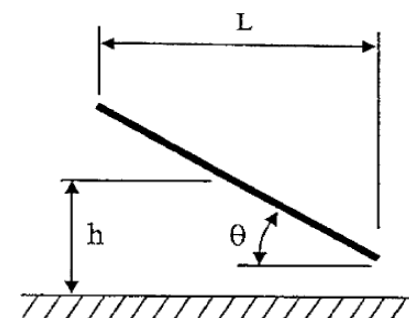
Notation:

L : horizontal dimension of roof, measured in the along wind direction, m

h : mean roof height, m

γ : direction of wind, degrees

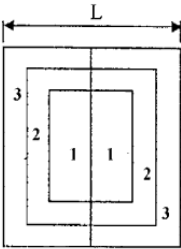
θ : angle of plane of roof from horizontal, degrees

Components and Cladding		0.25 ≤ h/L ≤ 1.0											
Figure 2.4.19A		Net Pressure Coefficient, C _N						Monoslope Free Roofs					
Open Buildings		θ ≤ 45°											
													
Roof Angle θ	Effective Wind Area	C _N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone3		Zone2		Zone 1		Zone3		Zone2		Zone 1	
0°	≤ a ²	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	>a ² , ≤4.0a ²	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	>4.0a ²	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	≤ a ²	3.2	-4.2	2.4	-2.1	1.6	-1.4	1.6	-5.1	0.5	-2.6	0.8	-1.7
	>a ² , ≤4.0a ²	2.4	-2.1	2.4	-2.1	1.6	-1.4	1.2	-2.6	1.2	-2.6	0.8	-1.7
	>4.0a ²	1.6	-1.4	1.6	-1.4	1.6	-1.4	0.8	-1.7	0.8	-1.7	0.8	-1.7
15°	≤ a ²	3.6	-3.8	2.7	-2.9	1.8	-1.9	2.4	-4.2	1.8	-3.2	1.2	-2.1
	>a ² , ≤4.0a ²	2.7	-2.9	2.7	-2.9	1.8	-1.9	1.8	-3.2	1.8	-3.2	1.2	-2.1
	>4.0a ²	1.8	-1.9	1.8	-1.9	1.8	-1.9	1.2	-2.1	1.2	-2.1	1.2	-2.3
30°	≤ a ²	5.2	-5	3.9	-3.8	2.6	-2.5	3.2	-4.6	2.4	-3.5	1.6	-2.3
	>a ² , ≤4.0a ²	3.9	-3.8	3.9	-3.8	2.6	-2.5	2.4	-3.5	2.4	-3.5	1.6	-2.3
	>4.0a ²	2.6	-2.5	2.6	-2.5	2.6	-2.5	1.6	-2.3	1.6	-2.3	1.6	-2.3
45°	≤ a ²	5.2	-4.6	3.9	-3.5	2.6	-2.3	4.2	-3.8	3.2	-2.9	2.1	-1.9
	>a ² , ≤4.0a ²	3.9	-3.5	3.9	-3.5	2.6	-2.3	3.2	-2.9	3.2	-2.9	2.1	-1.9
	>4.0a ²	2.6	-2.3	2.6	-2.3	2.6	-2.3	2.1	-1.9	2.1	-1.9	2.1	-1.9

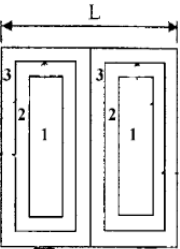
Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of e other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:
a : 10% of least horizontal dimension or 0.4h, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m
h : mean roof height, m
L : horizontal dimension of building, measured in along wind direction, m
θ: angle of plane of roof from horizontal, degrees

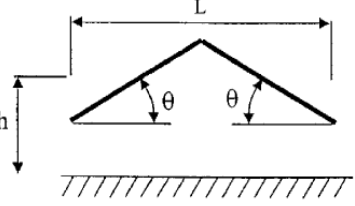
Components and Cladding		$0.25 \leq h/L \leq 1.0$	
Figure 2.4.19B	Net Pressure Coefficients, C_N	Monoslope Free Roofs	
Open Buildings		$\theta \leq 45^\circ$	



$\theta < 10^\circ$



$\theta \geq 10^\circ$

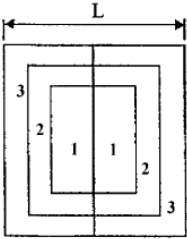


Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	2.2	-3.6	1.7	-1.8	1.1	-1.2	1	-5.1	0.8	-2.6	0.5	-1.7
	$>a^2, \leq 4.0a^2$	1.7	-1.8	1.7	-1.8	1.1	-1.2	0.8	-2.6	0.8	-2.6	0.5	-1.7
	$>4.0a^2$	1.1	-1.2	1.1	-1.2	1.1	-1.2	0.5	-1.7	0.5	-1.7	as	-1.7
15°	$\leq a^2$	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-3.2	0.8	-2.4	0.5	-1.6
	$>a^2, \leq 4.0a^2$	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	$>4.0a^2$	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
30°	$\leq a^2$	2.6	-1.8	2	-1.4	1.3	-0.9	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	2	-1.4	2	-1.4	1.3	-0.9	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.3	-0.9	1.3	-0.9	1.3	-0.9	0.5	-1.2	0.5	-1.2	0.5	-1.2
45°	$\leq a^2$	2.2	-1.6	1.7	-1.2	1.1	-0.8	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.7	-1.2	1.7	-1.2	1.1	-0.8	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.1	-0.8	1.1	-0.8	1.1	-0.8	0.5	-1.2	0.5	-1.2	0.5	-1.2

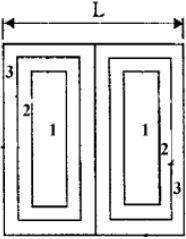
Notes:

- C_N denotes net pressures (contributions from top and bottom surfaces).
- Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
- For values of θ other than those shown, linear interpolation is permitted.
- Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
- Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
- Notation:
 - a : 10% of least horizontal dimension or 0.411, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m
 - h : mean roof height, m
 - L : horizontal dimension of building, measured in along wind direction, m
 - θ : angle of plane of roof from horizontal, degrees

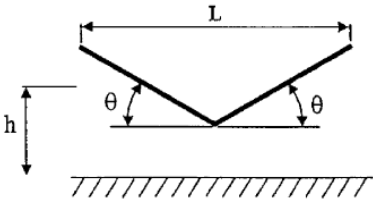
Components and Cladding		$0.25 \leq h/L \leq 1.0$	
Figure 2.4.19C	Net Pressure Coefficients, C_N	Troughed Free Roofs	
Open Buildings		$\theta \leq 45^\circ$	



$\theta < 10^\circ$



$\theta \geq 10^\circ$



Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.1	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-4.8	0.8	-2.4	0.5	-1.6
	$>a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
15°	$\leq a^2$	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
30°	$\leq a^2$	1.8	-2.6	1.4	-2	0.9	-1.3	1	-2.8	0.8	-2.1	0.5	-1.4
	$>a^2, \leq 4.0a^2$	1.4	-2	1.4	-2	0.9	-1.3	0.8	-2.1	0.8	-2.1	0.5	-1.4
	$>4.0a^2$	0.9	-1.3	1.9	-1.3	0.9	-1.3	0.5	-1.4	0.5	-1.4	0.5	-1.4
45°	$\leq a^2$	1.6	-2.2	1.2	-1.7	0.8	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.2	-1.7	1.2	-1.7	0.8	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	0.8	-1.1	1.8	-1.1	0.8	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2

Notes:

- C_N denotes net pressures (contributions from top and bottom surfaces).
- Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
- For values of θ other than those shown, linear interpolation is permitted.
- Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
- Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
- Notation:
 - a : 10% of least horizontal dimension or 0.411, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m
 - h : mean roof height, m
 - L : horizontal dimension of building, measured in along wind direction, m
 - θ : angle of plane of roof from horizontal, degrees

Other Structures-Method 2		All Height											
Figure 2.4.20	Force Coefficient, C_f	Solid Freestanding Walls & Solid Signs											
ELEVATION VIEW CROSS-SECTION VIEW CASE A CASE B CASE C PLAN VIEWS													
C_f , CASE A & CASE B													
Clearance Ratio, s/h	Aspect Ratio, B/s												
	≤ 0.05	0.1	0.2	0.5	1	2	4	5	10	20	30	≥ 45	
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35	1.35	1.30	1.30	1.30	1.30	
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45	1.45	1.40	1.40	1.40	1.40	
0.7	1.90	1.85	1.75	1.70	1.65	1.60	1.60	1.55	1.55	1.55	1.55	1.55	
0.5	1.95	1.85	1.80	1.75	1.75	1.70	1.70	1.70	1.70	1.70	1.70	1.75	
0.3	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.80	1.85	1.85	1.85	
0.2	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.85	1.90	1.90	1.95	
≤ 0.16	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95	
C_p , CASE C													
Region (horizontal distance from windward edge)	Aspect Ratio, B/s										Region (horizontal distance from windward edge)	Aspect Ratio, B/s	
	2	3	4	5	6	7	8	9	10	13		≥ 45	
0 to s	2.25	2.60	2.90	3.10*	3.30*	3.40*	3.55*	3.65*	3.75*	0 to s	4.00*	4.30*	
s to $2s$	1.50	1.70	1.90	2.00	2.15	2.25	2.30	2.35	2.45	s to $2s$	2.60	2.55	
$2s$ to $3s$		1.15	1.30	1.45	1.55	1.65	1.70	1.75	1.85	$2s$ to $3s$	2.00	1.95	
$3s$ to $10s$			1.10	1.05	1.05	1.05	1.05	1.00	0.95	$3s$ to $4s$	1.50	1.85	
										$4s$ to $5s$	1.35	1.85	
										$5s$ to $10s$	0.90	1.10	
										$>10s$	0.55	0.55	
*Values shall be multiplied by the following reduction factor when a return corner is present:		L_r/s	Reduction Factor										
		0.3	0.9										
		1.0	0.75										
		≥ 2	0.60										
PLAN VIEW OF WALL OR SIGN WITH A RETURN CORNER													
Notes:													
1. The term "signs" in notes below also applies to "freestanding walls".													
2. Signs with openings comprising less than 30% of the gross area are classified as solid signs. Force coefficients for solid signs with openings shall be permitted to be multiplied by the reduction factor $(1 - (1 - \epsilon)^{1.5})$.													
3. To allow for both normal and oblique wind directions, the following cases shall be considered:													
For $s/h < 1$:													
CASE A: resultant force acts normal to the face of the sign through the geometric center.													
CASE B: resultant force acts normal to the face of the sign at a distance from the geometric center toward the windward edge equal to 0.2 times the average width of the sign.													
For $B/s \geq 2$, CASE C must also be considered:													
CASE C: resultant forces act normal to the face of the sign through the geometric centers of each region.													
For $s/h = 1$:													
The same cases as above except that the vertical locations of the resultant forces occur at a distance above the geometric center equal to 0.05 times the average height of the sign.													
4. For CASE C where $s/h > 0.8$, force coefficients shall be multiplied by the reduction factor $(1.8 - s/h)$.													
5. Linear interpolation is permitted for values of s/h , B/s and L_r/s other than shown.													
6. Notation:													
B : horizontal dimension of sign, in meters;													
h : height of the sign, in meters;													
s : vertical dimension of the sign, in meters;													
ϵ : ratio of solid area to gross area;													
L_r : horizontal dimension of return corner, in meters													

Other Structures-Method 2		All Height		
Figure 2.4.21	Force Coefficient, C_f	Chimneys, Tanks, Rooftop Equipment, & Similar Structures		
Cross-Section	Type of Surface	h/D		
		1	7	25
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagonal)	All	1.0	1.1	1.5
Hexagonal or octagonal	All	1.0	1.2	1.4
Round $D\sqrt{q_z} > 5.3$, D in m, q_z in N/m^2	Moderately smooth	0.5	0.6	0.7
	Rough ($D'/D=0.02$)	0.7	0.8	0.9
	Very rough ($D'/D=0.08$)	0.8	1.0	0.2
Round $D\sqrt{q_z} \leq 5.3$, D in m, q_z in N/m^2	All	0.7	0.8	1.2

Notes:
The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
Linear interpolation is permitted for h/D values other than shown.

Notation:
 D : diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-section at elevation under consideration, in meters;
 D' : depth of protruding element such as ribs and spoilers, in meters;
 H : height of structure, meters and
 q_z : velocity pressure evaluated at height z above ground, in N/m^2

Other Structures-Method 2		All Height	
Figure 2.4.22	Force Coefficient, C_f	Open Signs & Lattice Frameworks	
ϵ	Flat-Sided Members	Rounded Members	
		$(D\sqrt{q_z} \leq 5.3,)$	$(D\sqrt{q_z} > 5.3,)$
	<0.1	2.0	1.2
	0.1 to 0.29	1.8	1.3
	0.3 to 0.7	1.6	1.5

Notes:

Signs with openings comprising 30% or more of the gross area are classified as open signs.

The calculation of the design wind forces shall be based on the area of all exposed members and elements projected on a plane normal to the wind direction. Forces shall be assumed to act parallel to the wind.

The area A_f consistent with these force coefficients is the solid area projected normal the wind direction.

Notation:

ϵ : ratio of solid area to gross area;

D : diameter of a typical round member, in meters

q_z : velocity pressure evaluated at height z above ground in N/m^2 .

Other Structures-Method 2		All Height
Figure 2.4.23	Force Coefficient, C_f	Trussed Tower
Open Structures		
Tower Cross Section		C_f
Square		$4.0 \epsilon^2 - 5.9 \epsilon + 4.0$
Triangle		$3.4 \epsilon^2 - 4.7 \epsilon + 3.4$

Notes:

- For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.
- The specified force coefficients are for towers with structural angles or similar flat-sided members.
- For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members: $0.51 \epsilon^2 + 0.57 \leq 1.0$
- Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:
 $1 + 0.75 \epsilon \leq 1.2$
- Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.
- Loads due to ice accretion as described in Section 11 shall be accounted for.
- Notation:

ϵ : ratio of solid area to gross area of one tower face for the segment under consideration.

Table 2.4.1: Basic Wind Speeds for Selected Locations in Bangladesh

Location	Basic Wind Speed (m/s)	Location	Basic Wind Speed (m/s)
Angarpota	47.8	Lalmonirhat	63.7
Bagerhat	77.5	Madaripur	68.1
Bandarban	62.5	Magura	65.0
Barguna	80.0	Manikganj	58.2
Barisal	78.7	Meherpur	58.2
Bhola	69.5	Maheshkhali	80.0
Bogra	61.9	Moulvibazar	53.0
Brahmanbaria	56.7	Munshiganj	57.1
Chandpur	50.6	Mymensingh	67.4
Chapai Nawabganj	41.4	Naogaon	55.2
Chittagong	80.0	Narail	68.6
Chuadanga	61.9	Narayanganj	61.1
Comilla	61.4	Narsinghdi	59.7
Cox's Bazar	80.0	Natore	61.9
Dahagram	47.8	Netrokona	65.6
Dhaka	65.7	Nilphamari	44.7
Dinajpur	41.4	Noakhali	57.1
Faridpur	63.1	Pabna	63.1
Feni	64.1	Panchagarh	41.4
Gaibandha	65.6	Patuakhali	80.0
Gazipur	66.5	Pirojpur	80.0
Gopalganj	74.5	Rajbari	59.1
Habiganj	54.2	Rajshahi	49.2
Hatiya	80.0	Rangamati	56.7
Ishurdi	69.5	Rangpur	65.3
Joypurhat	56.7	Satkhira	57.6
Jamalpur	56.7	Shariatpur	61.9
Jessore	64.1	Sherpur	62.5
Jhalakati	80.0	Sirajganj	50.6
Jhenaidah	65.0	Srimangal	50.6
Khagrachhari	56.7	St. Martin's Island	80.0
Khulna	73.3	Sunamganj	61.1
Kutubdia	80.0	Sylhet	61.1
Kishoreganj	64.7	Sandwip	80.0
Kurigram	65.6	Tangail	50.6
Kushtia	66.9	Teknaf	80.0
Lakshmipur	51.2	Thakurgaon	41.4

Importance Factor, I (Wind Loads)**Table 2.4.2**

Category or Importance Class	Non-Hurricane Prone Regions and Hurricane Prone Regions with $V = 38-44$ m/s	Hurricane Prone Regions with $V > 44$ m/s
I	0.87	0.77
II	1.0	1.00
III	1.15	1.15
IV	1.15	1.15

Note:

1. The building and structure classification categories are listed in Table 1.2.1

Terrain Exposure Constants**Table 2.4.3**

Exposure	α	Z_g (m)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (m)	$\bar{\epsilon}$	Z_{min} (m)*
A	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
B	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
C	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

* Z_{min} = minimum height used to ensure that the equivalent height z is greater of $0.6h$ or Z_{min} .

For buildings with $h \leq Z_{min}$, $\bar{z}$ shall be taken as Z_{min} .

Velocity Pressure Exposure Coefficients, K_z and K_d

Table 2.4.4

Height above ground level, z	Exposure (Note 1)			
	A		B	C
(m)	Case 1	Case 2	Case 1 & 2	Case 1 & 2
0-4.6	0.70	0.57	0.85	1.03
6.1	0.70	0.62	0.90	1.08
7.6	0.70	0.66	0.94	1.12
9.1	0.70	0.70	0.98	1.16
12.2	0.76	0.76	1.04	1.22
15.2	0.81	0.81	1.09	1.27
18	0.85	0.85	1.13	1.31
21.3	0.89	0.89	1.17	1.34
24.4	0.93	0.93	1.21	1.38
27.41	0.96	0.96	1.24	1.40
30.5	0.99	0.99	1.26	1.43
36.6	1.04	1.04	1.31	1.48
42.7	1.09	1.09	1.36	1.52
48.8	1.13	1.13	1.39	1.55
54.9	1.17	1.17	1.43	1.58
61.0	1.20	1.20	1.46	1.61
76.2	1.28	1.28	1.53	1.68
91.4	1.35	1.35	1.59	1.73
106.7	1.41	1.41	1.64	1.78
121.9	1.47	1.47	1.69	1.82
137.2	1.52	1.52	1.73	1.86
152.4	1.56	1.56	1.77	1.89

Notes:

1. Case 1: a. All components and cladding.
b. Main wind force resisting system in low-rise buildings designed using Figure 2.4.10.

Case 2: a. All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 2.4.10.
b. All main wind force resisting systems in other structures.

2. The velocity pressure exposure coefficient K_z may be determined from the following formula:
For $4.57\text{ m} \leq z \leq z_g$: $K_z = 2.01 (z/z_g)^{2/\alpha}$
For $z < 4.57\text{ m}$: $K_z = 2.01 (4.57/z_g)^{2/\alpha}$
Note: z shall not be taken less than 9.1 m for Case 1 in exposure A.

3. α and z_g are tabulated in Table 2.4.3.

4. Linear interpolation for intermediate values. of height z is acceptable.

5. Exposure categories are defined in 2.4.8.3.

Wind Directionality Factor, K_d **Table 2.4.5**

Structure Type	Directionality Factor K_d^*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

*Directionality Factor K_d has been calibrated with combinations of loads specified in Section 2.7. This factor shall only be applied when used in conjunction with load combinations specified in 2.7.4 and 2.7.5.

2.5 EARTHQUAKE LOADS

2.5.1 GENERAL

Minimum design earthquake forces for buildings, structures or components thereof shall be determined in accordance with the provisions of section 2.5. Some definitions and symbols relevant for earthquake resistant design for buildings are provided in Sections 2.5.2 and 2.5.3. Section 2.5.4 presents basic earthquake resistant design concepts. Section 2.5.5 describes procedures for soil investigations, while Section 2.5.6 describes procedures for determining earthquake ground motion for design. Section 2.5.7 describes different types of buildings and structural systems which possess different earthquake resistant characteristics. Static analysis procedures for design are described in Sections 2.5.8, 2.5.9 and 2.5.14. Dynamic analysis procedures are dealt with in Sections 2.5.10 to 2.5.13. Section 2.5.15 presents combination of earthquake loading effects in different directions and with other loading effects. Section 2.5.16 deals with allowable drift and deformation limits. Section 2.5.17 addresses design of non-structural components in buildings. Section 2.5.18 presents design considerations for buildings with seismic isolation systems. Design for soft storey condition in buildings is addressed in Section 2.5.19.

2.5.2 DEFINITIONS

The following definitions of terms shall be applicable only to the provisions of Section 2.5 :

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

BASE SHEAR : Total design lateral force or shear due to earthquake at the base of a structure.

BEARING WALL SYSTEM : A structural system without a complete vertical load carrying space frame.

BRACED FRAME : An essentially vertical truss system of the concentric or eccentric type provided to resist lateral forces.

BUILDING FRAME SYSTEM : An essentially complete space frame which provides support for gravity loads.

CAPACITY CURVE: A plot of the total applied lateral force, V_j , versus the lateral displacement of the control point, δ_j , as determined in a nonlinear static analysis.

CONTROL POINT: A point used to index the lateral displacement of the structure in a nonlinear static analysis.

CRITICAL DAMPING: Amount of damping beyond which the free vibration will not be oscillatory.

DAMPING: The effect of inherent energy dissipation mechanisms in a structure (due to sliding, friction, etc.) that results in reduction of effect of vibration, expressed as a percentage of the critical damping for the structure.

DESIGN ACCELERATION RESPONSE SPECTRUM: Smoothened idealized plot of maximum acceleration of a single degree of freedom structure as a function of structure period for design earthquake ground motion.

DESIGN EARTHQUAKE: The earthquake ground motion considered (for normal design) as two-thirds of the corresponding Maximum Considered Earthquake (MCE).

DIAPHRAGM: A horizontal or nearly horizontal system of structures acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes reinforced concrete floor slabs as well as horizontal bracing systems.

DUAL SYSTEM : A combination of a Special or Intermediate Moment Resisting Frame and Shear Walls or Braced Frames designed in accordance with the criteria of Sec 1.3.2.

DUCTILITY: Capacity of a structure, or its members to undergo large inelastic deformations without significant loss of strength or stiffness.

ECCENTRIC BRACED FRAME (EBF): A steel braced frame designed in conformance with Sec 10.20.15.

EPICENTRE: The point on the surface of earth vertically above the focus (point of origin) of the earthquake.

ESSENTIAL FACILITIES : Buildings and structures which are necessary to remain functional during an emergency or a post disaster period.

FLEXIBLE DIAPHRAGM : A floor or roof diaphragm shall be considered flexible, for purposes of this provision, when the maximum lateral deformation of the diaphragm is more than two times the average storey drift of the associated storey. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm under lateral load with the storey drift of adjoining vertical resisting elements under equivalent tributary lateral load.

FLEXIBLE ELEMENT OR SYSTEM : An element or system whose deformation under lateral load is significantly larger than adjoining parts of the system.

HORIZONTAL BRACING SYSTEM : A horizontal truss system that serves the same function as a floor or roof diaphragm.

IMPORTANCE FACTOR: It is a factor used to increase the design seismic forces for structures of importance.

INTENSITY OF EARTHQUAKE: It is a measure of the amount of ground shaking at a particular site due to an earthquake

INTERMEDIATE MOMENT RESISTING FRAME (IMRF): A concrete or steel frame designed in accordance with Sec 8.3 or 10.20.10 respectively.

LIQUEFACTION: State in saturated cohesionless soil wherein the effective shear strength is reduced to negligible value due to pore water pressure generated by earthquake vibrations, when the pore water pressure approaches the total confining pressure. In this condition, the soil tends to behave like a liquid.

MAGNITUDE OF EARTHQUAKE: The magnitude of earthquake is a number, which is a measure of energy released in an earthquake.

MAXIMUM CONSIDERED EARTHQUAKE (MCE): The most severe earthquake ground motion considered by this code.

MODAL MASS: part of the total seismic mass of the structure that is effective in mode k of vibration.

MODAL PARTICIPATION FACTOR: Amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions.

MODAL SHAPE COEFFICIENT: When a system is vibrating in a normal mode, at any particular instant of time, the vibration amplitude of mass i expressed as a ratio of the vibration amplitude of one of the masses of the system, is known as modal shape coefficient

MOMENT RESISTING FRAME : A frame in which members and joints are capable of resisting lateral forces primarily by flexure. Moment resisting frames are classified as ordinary moment resisting frames (OMRF), intermediate moment resisting frames (IMRF) and special moment resisting frames (SMRF).

NUMBER OF STOREYS (n) : Number of storeys of a building is the number of levels above the base. This excludes the basement storeys, where basement walls are connected with ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

ORDINARY MOMENT RESISTING FRAME (OMRF) : A moment resisting frame not meeting special detailing requirements for ductile behaviour.

P-DELTA EFFECT: It is the secondary effect on shears and moments of frame members due to action of the vertical loads due to the lateral displacement of building resulting from seismic forces.

PERIOD OF BUILDING: Fundamental period (for 1st mode) of vibration of building for lateral motion in direction considered.

RESPONSE REDUCTION FACTOR: It is the factor by which the actual base shear force that would develop if the structure behaved truly elastic during earthquake, is reduced to obtain design base shear. This reduction is allowed to account for the beneficial effects of inelastic deformation (resulting in energy dissipation) that can occur in a structure during a major earthquake, still ensuring acceptable response of the structure.

SEISMIC DESIGN CATEGORY: A classification assigned to a structure based on its importance factor and the severity of the design earthquake ground motion at the site.

SEISMIC-FORCE-RESISTING SYSTEM: That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces.

SHEAR WALL : A wall designed to resist lateral forces acting in its plane (sometimes referred to as a vertical diaphragm or a structural wall).

SOFT STOREY : Storey in which the lateral stiffness is less than 70 per cent of the stiffness of the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.

SITE CLASS: Site is classified based on soil properties of upper 30 meters.

SPACE FRAME : A three-dimensional structural system without bearing walls composed of members interconnected so as to function as a complete self contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

SPECIAL MOMENT RESISTING FRAME (SMRF) : A moment resisting frame specially detailed to provide ductile behaviour complying with the seismic requirements provided in Chapters 8 and 10 for concrete and steel frames respectively.

STOREY : The space between consecutive floor levels. Storey-x is the storey below level-x.

STOREY SHEAR : The total horizontal shear force at a particular storey (level).

STOREY DRIFT: The horizontal deflection at the top of the story relative to bottom of the storey.

STRENGTH : The usable capacity of an element or a member to resist the load as prescribed in these provisions.

TARGET DISPLACEMENT: An estimate of the maximum expected displacement of the control point calculated for the design earthquake ground motion in nonlinear static analysis.

VERTICAL LOAD-CARRYING FRAME : A space frame designed to carry all vertical gravity loads.

WEAK STOREY : Storey in which the lateral strength is less than 80 per cent of that of the storey above.

2.5.3 SYMBOLS AND NOTATION

The following symbols and notation shall apply to the provisions of this section :

A_x	=	torsion amplification factor at level-x.
C_d	=	Deflection amplification factor.
C_s	=	Normalized acceleration response spectrum.
C_t	=	numerical coefficient to determine building period
e_{oi}	=	Accidental eccentricity of floor mass at level-i
F_y, F_{ny}, F_x	=	design lateral force applied to level-i, -n, or -x respectively.
	=	lateral forces on an element or component or on equipment supports.
g	=	acceleration due to gravity.
h_y, h_{ny}, h_x	=	Height in metres above the base to level i, -n or -x respectively
h_{sx}	=	Storey Height of storey x (below level-x)
I	=	Importance factor
<i>Level-i</i>	=	floor level of the structure referred to by the subscript i, e.g., i = 1 designates the first level above the base.
<i>Level-n</i>	=	Uppermost level in the main portion of the structure.
M_x	=	Overtaking moment at level-x
N_i	=	Standard Penetration Number of soil layer i
P_x	=	Total vertical design load at level-x
R	=	Response reduction factor for structural systems
S	=	Soil factor.
S_a	=	Design Spectral Acceleration (in units of g)
S_{ui}	=	Undrained shear strength of cohesive layer i
T	=	Fundamental period of vibration of structure, in seconds, of the structure in the direction under consideration.
T_e	=	Effective fundamental period of the structure in the direction under consideration, as determined for nonlinear static analysis

V	=	Total design base shear calculated by equivalent static analysis
V_1	=	Total applied lateral force at the first increment of lateral load in nonlinear static analysis.
V_y	=	Effective yield strength determined from a bilinear curve fitted to the capacity curve
V_{rs}	=	Total design base shear calculated by response spectrum analysis
V_{th}	=	Total design base shear calculated by time history analysis
V_{si}	=	Shear wave velocity of soil layer i
V_x	=	Design storey shear in storey x
W	=	Total seismic weight of building
w_i, w_x	=	Portion of W which is assigned to level i and x respectively
Z	=	Seismic zone coefficient.
δ_i	=	Horizontal displacement at level-i relative to the base due to applied lateral forces.
δ_j	=	The displacement of the control point at load increment j.
δ_T	=	The target displacement of the control point.
δ_1	=	The displacement of the control point at the first increment of lateral load.
δ_y	=	The effective yield displacement of the control point determined from a bilinear curve fitted to the capacity curve
Δ_a	=	Maximum allowable storey drift
Δ_x	=	Design storey drift of storey x
η	=	Damping correction factor
ϕ_{ik}	=	Modal shape coefficient at level i for mode k
θ	=	Stability coefficient to assess P-delta effects
ξ	=	Viscous damping ratio of the structure

2.5.4 EARTHQUAKE RESISTANT DESIGN – BASIC CONCEPTS

2.5.4.1 GENERAL PRINCIPLES

The purpose of earthquake resistant design provisions in this code is to provide guidelines for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. It is not economically feasible to design and construct buildings without any damage for a major earthquake event. The intent is therefore to allow inelastic deformation and structural damage at

preferred locations in the structure without endangering structural integrity and to prevent structural collapse during a major earthquake.

The expected earthquake ground motion at the site due to all probable earthquakes may be evaluated in deterministic or probabilistic terms. The ground motion at the site due to an earthquake is a complex phenomena and depends on several parameters such as earthquake magnitude, focal depth, earthquake source characteristics, distance from earthquake epicenter, wave path characteristics, as well as local soil conditions at the site. The seismic zoning map divides the country into four seismic zones with different expected levels of intensity of ground motion. Each seismic zone has a zone coefficient which provides expected peak ground acceleration values on rock/firm soil corresponding to the maximum considered earthquake (MCE). The design basis earthquake is taken as $2/3$ of the maximum considered earthquake.

The effects of the earthquake ground motion on the structure is expressed in terms of an idealized elastic design acceleration response spectrum, which depends on (a) seismic zone coefficient and local soil conditions defining ground motion and (b) importance factor and response reduction factor representing building considerations. The earthquake forces acting on the structure is reduced using the response modification/reduction factor R in order to take advantage of the inelastic energy dissipation due to inherent ductility and redundancy in the structure as well as material over-strength. The importance factor I increases design forces for important structures. If suitable lateral force resisting systems with adequate ductility and detailing and good construction are provided, the building can be designed for a response reduction factor R which may be as high as 5 to 8. Because of this fact, the provisions of this Code for ductility and detailing need to be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake. The elastic deformations calculated under these reduced design forces are multiplied by the deflection amplification factor, C_d , to estimate the deformations likely to result from the design earthquake.

The seismic design guidelines presented in this section are based on the assumption that the soil supporting the structure will not liquefy, settle or slide due to loss of strength during the earthquake. Reinforced and prestressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur. Ductile detailing of reinforced concrete members is of prime importance. In steel structures, members and their connections should be so proportioned that high ductility is obtained, avoiding premature failure due to elastic or inelastic buckling of any type.

The building structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a building structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the building structure, shall be established in accordance with one of the applicable procedures indicated in Section 2.5 and the corresponding internal forces and deformations in the members of the structure shall be determined. The deformation of the structure shall not exceed the prescribed limits under the action of the design seismic forces.

2.5.4.2 CHARACTERISTICS OF EARTHQUAKE RESISTANT BUILDINGS

The desirable characteristics of earthquake resistant buildings are described below:

Structural Simplicity, Uniformity and Symmetry:

Structural simplicity, uniformity and plan symmetry is characterized by an even distribution of mass and structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building to its foundation. The modelling, analysis, detailing and construction of simple (regular) structures are subject to much less uncertainty, hence the prediction of its seismic behaviour is much more reliable.

A building configuration with symmetrical layout of structural elements of the lateral force resisting system, and well-distributed in-plan, is desirable. Uniformity along the height of the building is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might cause premature collapse.

Some basic guidelines are given below:

- (i) With respect to the lateral stiffness and mass distribution, the building structure shall be approximately symmetrical in plan with respect to two orthogonal axes.
- (ii) Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.
- (iii) All structural elements of the lateral load resisting systems, such as cores, structural walls, or frames shall run without interruption from the foundations to the top of the building.
- (iv) An irregular building may be subdivided into dynamically independent regular units well separated against pounding of the individual units to achieve uniformity.
- (v) The length to breadth ratio ($\lambda = L_{\max}/L_{\min}$) of the building in plan shall not be higher than 4, where $L_{\max}$ and $L_{\min}$ are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions.

Structural Redundancy:

A high degree of redundancy accompanied by redistribution capacity through ductility is desirable, enabling a more widely spread energy dissipation across the entire structure and an increased total dissipated energy. The use of evenly distributed structural elements increases redundancy. Structural systems of higher static indeterminacy may result in higher response reduction factor R .

Horizontal Bi-directional Resistance and Stiffness:

Horizontal earthquake motion is a bi-directional phenomenon and thus the building structure needs to resist horizontal action in any direction. The structural elements of lateral force resisting system should be arranged in an orthogonal (in plan) pattern, ensuring similar resistance and stiffness characteristics in both main directions. The stiffness characteristics of the structure should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.

Torsional Resistance and Stiffness

Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different

structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

Diaphragm Behaviour

In buildings, floors (including the roof) act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together.

Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure.

The in-plane stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements.

Foundation

The design and construction of the foundation and of its connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation. For buildings with individual foundation elements (footings or piles), the use of a foundation slab or tie-beams between these elements in both main directions is recommended, as described in Chapter 3.

2.5.5 INVESTIGATION AND ASSESSMENT OF SITE CONDITIONS

2.5.5.1 SITE INVESTIGATION

Appropriate site investigations should be carried out to identify the ground conditions influencing the seismic action.

The ground conditions at the building site should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification during an earthquake. The possibility of such phenomena should be investigated in accordance with standard procedures described in Section 3.

The intent of the site investigation is to classify the Site into one of types SA, SB, SC, SD, SE, S_1 and S_2 as defined in Section 2.5.5.2. Such classification is based on site profile and evaluated soil properties (shear wave velocity, Standard Penetration Resistance, undrained shear strength, soil type). The site class is used to determine the effect of local soil conditions on the earthquake ground motion.

For sites representing special soil type S_1 or S_2 , site specific special studies for the ground motion should be done. Soil type S_1 , having very low shear wave velocity and low material damping, can produce anomalous seismic site amplification and soil-structure interaction effects. For S_2 soils, possibility of soil failure should be studied.

For a structure belonging to Seismic Design Category C or D (Section 2.5.7.2), site investigation should also include determination of soil parameters for the assessment of the following:

- a. Slope instability.
- b. Potential for Liquefaction and loss of soil strength.
- c. Differential settlement.
- d. Surface displacement due to faulting or lateral spreading.
- e. Lateral pressures on basement walls and retaining walls due to earthquake ground motion.

Liquefaction potential and possible consequences should be evaluated for design earthquake ground motions consistent with peak ground accelerations. Any Settlement due to densification of loose granular soils under design earthquake motion should be studied. The occurrence and consequences of geologic hazards such as slope instability or surface faulting should also be considered. The dynamic lateral earth pressure on basement walls and retaining walls during earthquake ground shaking is to be considered as an earthquake load for use in design load combinations

2.5.5.2 SITE CLASSIFICATION

Site will be classified as type SA, SB, SC, SD, SE, S₁ and S₂ based on the provisions of this section. Classification will be done in accordance with Table 2.5.1 based on the soil properties of upper 30 meters of the site profile.

Average soil properties will be determined as given in the following equations:

$$\bar{V}_s = \sum_{i=1}^n d_i / \sum_{i=1}^n \frac{d_i}{V_{si}} \quad (2.5.1)$$

$$\bar{N} = \sum_{i=1}^n d_i / \sum_{i=1}^n \frac{d_i}{N_i} \quad (2.5.2)$$

$$\bar{S}_u = \sum_{i=1}^k d_{ci} / \sum_{i=1}^k \frac{d_{ci}}{S_{ui}} \quad (2.5.3)$$

where,

n = number of soil layers in upper 30 m

d_i = thickness of layer i

V_{si} = shear wave velocity of layer i

N_i = Field (uncorrected) Standard Penetration Value for layer i

k = number of cohesive soil layers in upper 30 m

d_{ci} = thickness of cohesive layer i

S_{ui} = Undrained shear strength of cohesive layer i

The site profile up to a depth of 30 m is divided into n number of distinct soil or rock layers. Where some of the layers are cohesive, k is the number of cohesive layers. Hence $\sum_{i=1}^n d_i = 30$ m, while $\sum_{i=1}^k d_{ci} < 30$

m if $k < n$ in other words if there are both cohesionless and cohesive layers. The standard penetration value N as directly measured in the field without correction will be used.

The site classification should be done using average shear wave velocity $\bar{V}_s$ if this can be estimated, otherwise the value of $\bar{N}$ may be used.

Table 2.5.1: Site classification based on soil properties

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity $\bar{V}_s$ (m/s)	Standard Penetration Value, $\bar{N}$ (blows/30cm)	Undrained shear strength, $\bar{S}_u$ (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
SE	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_s > 800$ m/s.	--	--	--
S ₁	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ($PI > 40$) and high water content	< 100 (indicative)	--	10 - 20
S ₂	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S ₁	--	--	--

2.5.6 EARTHQUAKE GROUND MOTION

2.5.6.1 REGIONAL SEISMICITY

Bangladesh can be affected by moderate to strong earthquake events due to its proximity to the collision boundary of the Northeast moving Indian plate and Eurasian Plate. Strong historical earthquakes with magnitude greater than 7.0 have affected parts of Bangladesh in the last 150 years, some of them had

their epicenters within the country. A brief description of the local geology, tectonic features and earthquake occurrence in the region is given in Appendix C.

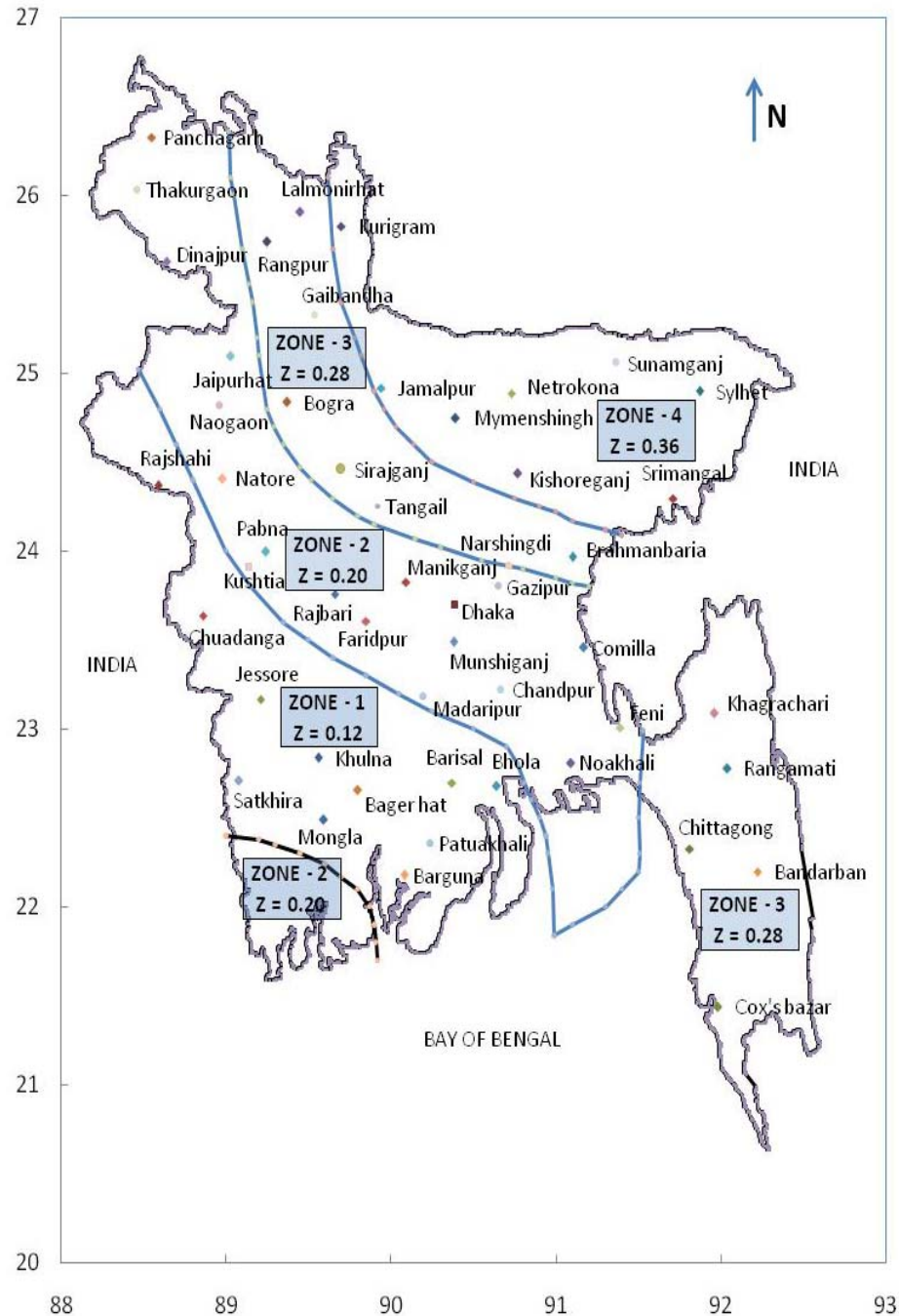


Fig. 2.5.1: Seismic Zoning Map of Bangladesh

2.5.6.2 SEISMIC ZONING

The intent of the seismic zoning map is to give an indication of the Maximum Considered Earthquake (MCE) motion at different parts of the country. In probabilistic terms, the MCE motion may be considered to correspond to having a 2% probability of exceedance within a period of 50 years. The country has been

divided into four seismic zones with different levels of ground motion. Table 2.5.2 includes a description of the four seismic zones. Fig. 2.5.1 presents a map of Bangladesh showing the boundaries of the four zones. Each zone has a seismic zone coefficient (Z) which represents the maximum considered peak ground acceleration (PGA) on very stiff soil/rock (site class SA) in units of g (acceleration due to gravity). The zone coefficients (Z) of the four zones are: Z=0.12 (Zone 1), Z=0.20 (Zone 2), Z=0.28 (Zone 3) and Z=0.36 (Zone 4). Table 2.5.3 lists zone coefficients for some important towns of Bangladesh. The most severe earthquake prone zone, Zone 4 is in the northeast which includes Sylhet and has a maximum PGA value of 0.36g. Dhaka city falls in the moderate seismic intensity zone with Z=0.2, while Chittagong city falls in a severe intensity zone with Z=0.28.

Table 2.5.2 Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Table 2.5.3 Seismic Zone Coefficient Z for Some Important Towns of Bangladesh

Town	Z	Town	Z	Town	Z	Town	Z
Bagerhat	0.12	Dinajpur	0.20	Kushtia	0.20	Panchagarh	0.20
Bandarban	0.28	Faridpur	0.20	Lalmanirhat	0.28	Patuakhali	0.12
Barguna	0.12	Feni	0.20	Madaripur	0.20	Rajbari	0.20
Barisal	0.12	Gaibandha	0.28	Manikganj	0.20	Rajshahi	0.12
Bhola	0.12	Gazipur	0.20	Mongla	0.12	Rangamati	0.28
Bogra	0.28	Habiganj	0.36	Munshiganj	0.20	Rangpur	0.28
Brahmanbaria	0.28	Jaipurhat	0.20	Mymensingh	0.36	Satkhira	0.12
Chandpur	0.20	Jamalpur	0.36	Narsingdi	0.28	Sirajganj	0.28
Chittagong	0.28	Jessore	0.12	Natore	0.20	Srimangal	0.36
Chuadanga	0.12	Khagrachari	0.28	Naogaon	0.20	Sunamganj	0.36
Comilla	0.20	Khulna	0.12	Netrakona	0.36	Sylhet	0.36
Cox's Bazar	0.28	Kishoreganj	0.36	Noakhali	0.20	Tangail	0.28
Dhaka	0.20	Kurigram	0.36	Pabna	0.20	Thakurgaon	0.20

2.5.6.3 DESIGN RESPONSE SPECTRUM

The earthquake ground motion for which the building has to be designed is represented by the design response spectrum. Both static and dynamic analysis methods are based on this response spectrum. This spectrum represents the spectral acceleration for which the building has to be designed as a function of the building period, taking into account the ground motion intensity. The spectrum is based on elastic analysis but in order to account for energy dissipation due to inelastic deformation and benefits of structural redundancy, the spectral accelerations are reduced by the response modification factor R . For important structures, the spectral accelerations are increased by the importance factor I . The design basis earthquake (DBE) ground motion is selected at a ground shaking level that is 2/3 of the maximum considered earthquake (MCE) ground motion. The effect of local soil conditions on the response spectrum is incorporated in the normalized acceleration response spectrum C_s .

The spectral acceleration for the design earthquake is given by the following equation:

$$S_a = \frac{2}{3} \frac{ZI}{R} C_s \quad (2.5.4)$$

where,

S_a = Design spectral acceleration (in units of g), which shall not be less than $2/3 \cdot ZI \cdot \beta$.

β = coefficient used to calculate lower bound for S_a . Recommended value for β is 0.2.

Z = Seismic zone coefficient, as defined in Section 2.5.6.2

I = Structure importance factor, as defined in Section 2.5.7.1

R = Response reduction factor which depends on the type of structural system given in Table 2.5.7. The ratio I/R cannot be greater than one.

C_s = Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) as defined by Equations 2.5.5a-d

$$C_s = S \left(1 + \frac{T}{T_B} (2.5\eta - 1) \right) \quad \text{for } 0 \leq T \leq T_B \quad (2.5.5a)$$

$$C_s = 2.5S\eta \quad \text{for } T_B \leq T \leq T_C \quad (2.5.5b)$$

$$C_s = 2.5S\eta \left(\frac{T_C}{T} \right) \quad \text{for } T_C \leq T \leq T_D \quad (2.5.5c)$$

$$C_s = 2.5S\eta \left(\frac{T_C T_D}{T^2} \right) \quad \text{for } T_D \leq T \leq 4 \text{ sec} \quad (2.5.5d)$$

C_s depends on S and values of T_B , T_C and T_D , (Fig. 2.5.2) which are all functions of the site class. Constant C_s value between periods T_B and T_C represents constant spectral acceleration.

S = Soil factor which depends on site class and is given in Table 2.5.4

T = Structure (building) period as defined in Section 2.5.9.2

T_B = Lower limit of the period of the constant spectral acceleration branch given in Table 2.5.4 as a function of site class.

T_C = Upper limit of the period of the constant spectral acceleration branch given in Table 2.5.4 as a function of site class

T_D = Lower limit of the period of the constant spectral displacement branch given in Table 2.5.4 as a function of site class

η = Damping correction factor as a function of damping with a reference value of $\eta=1$ for 5% viscous damping. It is given by the following expression:

$$\eta = \sqrt{10 / (5 + \xi)} \geq 0.55 \quad (2.5.6)$$

where, ξ is the viscous damping ratio of the structure, expressed as a percentage of critical damping. The value of η cannot be smaller than 0.55.

Figure 2.5.2 : Typical Shape of the Elastic Response Spectrum Coefficient C_s

Table 2.5.4 : Site dependent soil factor and other parameters defining elastic response spectrum

Soil type	S	$T_B(s)$	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

The anticipated (design basis earthquake) peak ground acceleration (PGA) for rock or very stiff soil (site class SA) is $2/3 \cdot Z$. However, for design, the ground motion is modified through the use of response reduction factor R and importance factor I , resulting in $PGA_{rock} = 2/3 \cdot Z \cdot I / R$. Fig. 2.5.3 shows the normalized acceleration response spectrum C_s for 5% damping, which may be defined as the 5% damped spectral acceleration (obtained by Eq.2.5.4) normalized with respect to PGA_{rock} . This figure demonstrates the significant influence of site class on the response spectrum.

Figure 2.5.3: Normalized Design Acceleration Response Spectrum for different site classes.

Design Spectrum for Elastic Analysis

For site classes S_A to S_E , the design acceleration response spectrum for elastic analysis methods is obtained using Eq.(2.5.4) to compute S_a (in units of g) as a function of period T . The design acceleration response spectrum represents the expected ground motion (Design Basis Earthquake) divided by the factor R/I .

Design Spectrum for Inelastic Analysis

For inelastic analysis methods, the anticipated ground motion (Design Basis Earthquake) is directly used. Corresponding real design acceleration response spectrum is used, which is obtained by using $R=1$ and $I=1$ in Eq.(2.5.4). The 'real design acceleration response spectrum' is equal to 'design acceleration response spectrum' multiplied by R/I .

Site-Specific Design Spectrum

For site class S_1 and S_2 , site-specific studies are needed to obtain design response spectrum. For important projects, site-specific studies may also be carried out to determine spectrum instead of using Eq.(2.5.4). The objective of such site-specific ground-motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using simplified equations.

2.5.7 BUILDING CATEGORIES

2.5.7.1 IMPORTANCE FACTOR

Buildings are classified in four occupancy categories in Chapter 1 (Table 1.2.1), depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. Depending on occupancy category, buildings may be designed for higher seismic forces using importance factor

greater than one. Table 2.5.5 defines different occupancy categories and corresponding importance factor.

Table 2.5.5 Importance Factors for Buildings and Structures for Earthquake design

Occupancy Category	Importance factor I
I or II	1.0
III	1.25
IV	1.5

2.5.7.2 SEISMIC DESIGN CATEGORY

Buildings shall be assigned a seismic design category among B, C or D based on seismic zone, local site conditions and importance class of building, as given in Table 2.5.6. Seismic design category D has the most stringent seismic design detailing, while seismic design category B has the least seismic design detailing requirements.

Table 2.5.6 Seismic Design Category of Buildings

	Occupancy Category I, II and III				Occupancy Category IV			
Site Class	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S ₁ , S ₂	D	D	D	D	D	D	D	D

2.5.7.3 BUILDING IRREGULARITY:

Buildings with irregularity in plan or elevation suffer much more damage in earthquakes than buildings with regular configuration. A building may be considered as irregular, if at least one of the conditions given below are applicable:

2.5.7.3.1 Plan Irregularity

i) Torsion irregularity

To be considered for rigid floor diaphragms, when the maximum storey drift ($\Delta_{\max}$) as shown in Fig.2.5.4a, computed including accidental torsion, at one end of the structure is more than 1.2 times the average ($\Delta_{\text{avg}} = (\Delta_{\max} + \Delta_{\min})/2$) of the storey drifts at the two ends of the structure. If $\Delta_{\max} > 1.4\Delta_{\text{avg}}$ then the irregularity is termed as extreme torsional irregularity.

ii) Re-entrant corners

Both projections of the structure beyond a re-entrant corner (Fig.2.5.4b) are greater than 15 percent of its plan dimension in the given direction.

iii) Diaphragm Discontinuity

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out (Fig.2.5.4c) or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one storey to the next.

iv) Out- Of-Plane Offsets

Discontinuities in a lateral force resistance path, such as out of-plane offsets of vertical elements, as shown in Fig. 2.5.4d.

v) Non-parallel Systems

The vertical elements resisting the lateral force are not parallel to or symmetric (Fig. 2.5.4e) about the major orthogonal axes of the lateral force resisting elements.

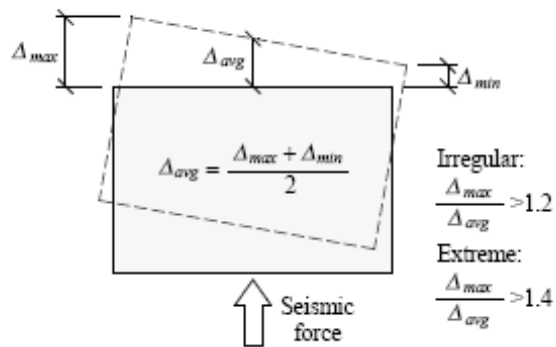


Fig. 2.5.4a Torsional Irregularity

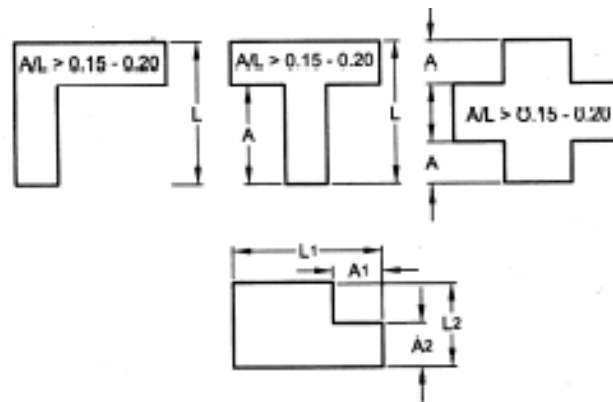


Fig.2.5.4b Re-entrant corners ($A/L > 0.15$)

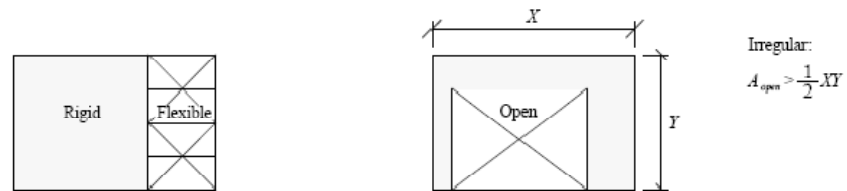


Fig.2.5.4c Diaphragm Discontinuity

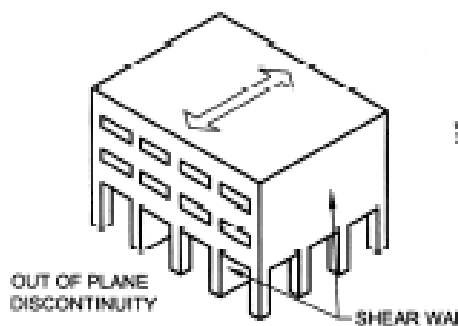


Fig.2.5.4d Out- Of-Plane Offsets of Shear Wall



Fig.2.5.4e Non-parallel Systems of Shear Wall

2.5.7.3.2 Vertical Irregularity

i) Stiffness Irregularity — Soft Storey

A soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three storeys above irregularity (Fig. 2.5.5a). An extreme soft storey is defined where its lateral stiffness is less than 60% of that in the storey above or less than 70% of the average lateral stiffness of the three storeys above.

ii) Mass Irregularity

The seismic weight of any storey is more than twice of that of its adjacent storeys (Fig. 2.5.5b). This irregularity need not be considered in case of roofs.

iii) Vertical Geometric Irregularity

This irregularity exists for buildings with setbacks with dimensions given in Fig.2.5.5c.

iv) Vertical In-Plane Discontinuity in Vertical Elements Resisting Lateral Force

An in-plane offset of the lateral force resisting elements greater than the length of those elements (Fig. 2.5.5d).

v) Discontinuity in Capacity — Weak Storey

A weak storey is one in which the storey lateral strength is less than 80% of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction (Fig. 2.5.5e). An extreme weak storey is one where the storey lateral strength is less than 65% of that in the storey above.

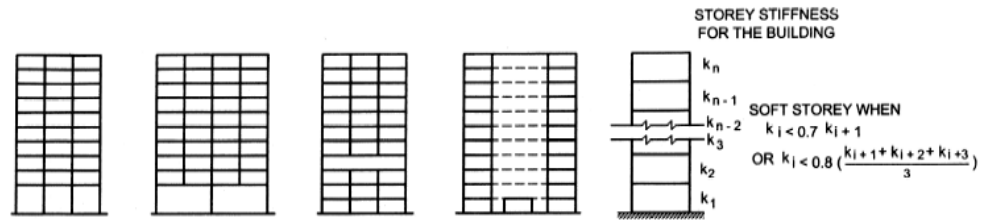


Fig. 2.5.5a Soft Storey

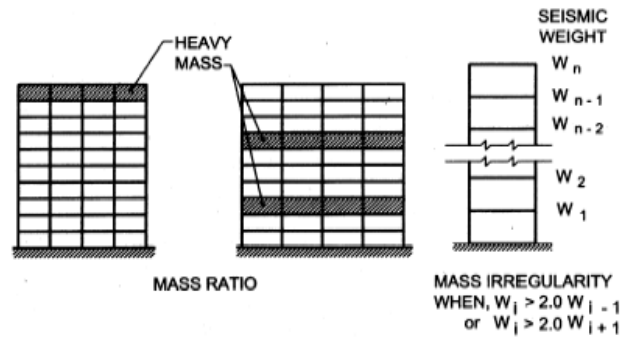


Fig. 2.5.5b Mass Irregularity

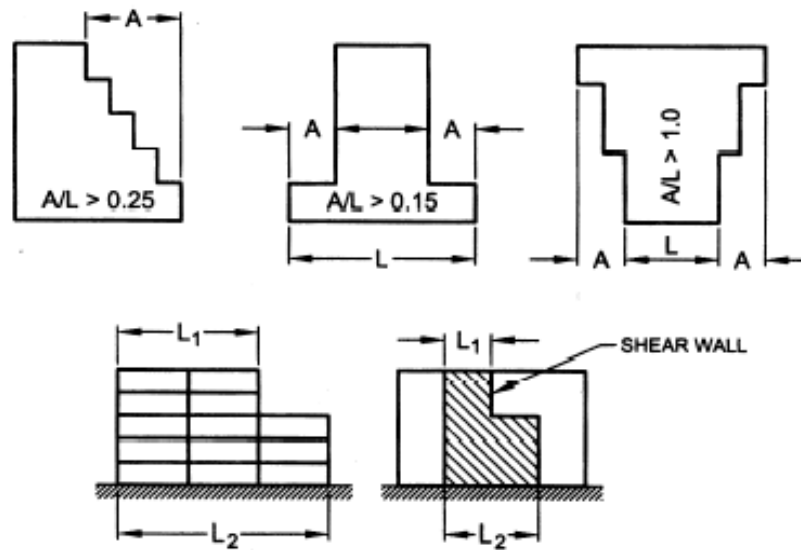


Fig.2.5.5c Vertical Geometric Irregularity (Setback Structures)

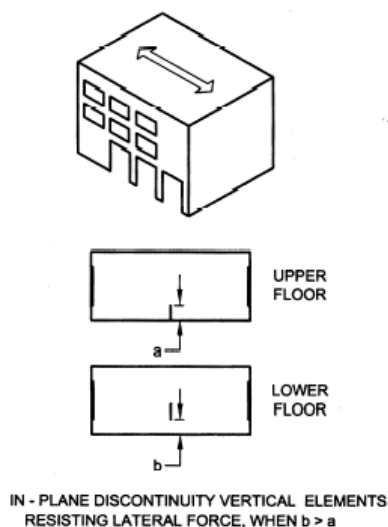


Fig.2.5.5d Vertical In-Plane Discontinuity in Vertical Elements Resisting Lateral Force

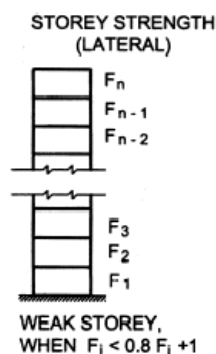


Fig.2.5.5e Weak Storey

2.5.7.4 TYPE OF STRUCTURAL SYSTEMS

The basic lateral and vertical seismic force-resisting system shall conform to one of the types A to G indicated in Table 2.5.7. Each type is again subdivided by the types of vertical elements used to resist lateral seismic forces. A combination of systems may also be permitted as stated in Section 2.5.7.5.

The structural system to be used shall be in accordance with the seismic design category indicated in Table 2.5.6. Structural systems that are not permitted for a certain seismic design category are indicated by “NP”. Structural systems that do not have any height restriction are indicated by “NL”. Where there is height limit, the maximum height in meters is given.

The response reduction factor, R , and the deflection amplification factor, C_d , indicated in Table 2.5.7 shall be used in determining the design base shear and design story drift.

The selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system.

Table 2.5.7 Response reduction factor, deflection amplification factor for different Structural Systems and height limitations (m) for different seismic design categories

Seismic Force-Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Seis. Design Category B	Seis. Design Category C	Seis. Design Category D	Height limit (m)
A. BEARING WALL SYSTEMS (no frame)						
1. Special reinforced concrete shear walls	5	5	NL	NL	50	
2. Ordinary reinforced concrete shear walls	4	4	NL	NL	NP	
3. Ordinary reinforced masonry shear walls	2	1.75	NL	50	NP	
4. Ordinary plain masonry shear walls	1.5	1.25	18	NP	NP	
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)						
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8	4	NL	NL	50	
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	4	NL	NL	50	
3. Special steel concentrically braced frames	6	5	NL	NL	50	
4. Ordinary steel concentrically braced frames	3.25	3.25	NL	NL	11	
5. Special reinforced concrete shear walls	6	5	NL	50	50	
6. Ordinary reinforced concrete shear walls	5	4.25	NL	NL	NP	
7. Ordinary reinforced masonry shear walls	2	2	NL	50	NP	
8. Ordinary plain masonry shear walls	1.5	1.25	18	NP	NP	
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)						
1. Special steel moment frames	8	5.5	NL	NL	P	
2. Intermediate steel moment frames	4.5	4	NL	NL	35	
3. Ordinary steel moment frames	3.5	3	NL	NL	NP	
4. Special reinforced concrete moment frames	8	5.5	NL	NL	NL	
5. Intermediate reinforced concrete moment frames	5	4.5	NL	NL	NP	
6. Ordinary reinforced concrete moment frames	3	2.5	NL	NP	NP	
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Steel eccentrically braced frames	8	4	NL	NL	NL	
2. Special steel concentrically braced frames	7	5.5	NL	NL	NL	

Seismic Force-Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Height limit (m)		
			Seis. Design Category B	Seis. Design Category C	Seis. Design Category D
3. Special reinforced concrete shear walls	7	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	5	NL	NL	NP
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)					
1. Special steel concentrically braced frames	6	5	NL	NL	11
2. Special reinforced concrete shear walls	6.5	5	NL	NL	50
3. Ordinary reinforced masonry shear walls	3	3	NL	50	NP
4. Ordinary reinforced concrete shear walls	5.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	4	NL	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE	3	3	NL	NL	NP
NOTE:					
1. Under seismic design category, NL = No height restriction, NP = Not permitted, Number represents maximum allowable height (m).					
2. Dual Systems include buildings which consist of both moment resisting frame and shear walls (or braced frame) where both systems resist the total design forces in proportion to their lateral stiffness.					
3. See Table 10.C.3.1 of Sec. 10 for additional values of R and C_d and height limits for some other types of steel structures not covered in Table 2.5.7					

Seismic force-resisting systems that are not given in Table 2.5.7 may be permitted if substantial analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 2.5.7 for equivalent response modification coefficient, R , and deflection amplification factor, C_d , values.

2.5.7.5 COMBINATION OF STRUCTURAL SYSTEMS

2.5.7.5.1 Combinations of Structural Systems in Different Directions:

Different seismic force-resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R and C_d coefficients shall apply to each system, including the limitations on system use contained in Table 2.5.7.

2.5.7.5.2 Combinations of Structural Systems in the Same Direction:

Where different seismic force-resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as dual systems, the

more stringent system limitation contained in Table 2.5.7 shall apply. The value of R used for design in that direction shall not be greater than the least value of R for any of the systems utilized in that direction. The deflection amplification factor, C_d in the direction under consideration at any story shall not be less than the largest value of this factor for the R factor used in the same direction being considered

2.5.8 STATIC ANALYSIS PROCEDURE

Although analysis of buildings subjected to dynamic earthquake loads should theoretically require dynamic analysis procedures, for certain type of building structures subjected to earthquake shaking, simplified static analysis procedures may also provide reasonably good results. The equivalent static force method is such a procedure for determining the seismic lateral forces acting on the structure. This type of analysis may be applied to buildings whose seismic response is not significantly affected by contributions from modes higher than the fundamental mode in each direction. This requirement is deemed to be satisfied in buildings which fulfill the following two conditions:

- a. The building period in the two main horizontal directions is smaller than both $4T_c$ (T_c is defined in Section 2.5.6.3) and 2 sec.
- b. The building doesn't possess irregularity in elevation as defined in Section 2.5.7.3.

2.5.9 EQUIVALENT STATIC ANALYSIS

The evaluation of the seismic loads starts with the calculation of the design base shear which is derived from the design response spectrum presented in Section 2.5.6.3. This section presents different computations relevant to the equivalent static analysis procedure.

2.5.9.1 DESIGN BASE SHEAR

The seismic design base shear force in a given direction shall be determined from the following relation:

$$V = S_a W \quad (2.5.7)$$

where,

S_a = Lateral seismic force coefficient calculated using Eq.2.5.4 (Section 2.5.6.3). It is the design spectral acceleration (in units of g) corresponding to the building period T (computed as per Section 2.5.9.2).

W = Total seismic weight of the building defined in Section 2.5.9.3

2.5.9.2 BUILDING PERIOD

The fundamental period T of the building in the horizontal direction under consideration shall be determined using the following guidelines:

- a) Structural dynamics procedures (such as Rayleigh method or modal eigenvalue analysis), using structural properties and deformation characteristics of resisting elements, may be used to determine the fundamental period T of the building in the direction under consideration. This period shall not exceed the approximate fundamental period determined by Equation (2.5.8) by more than 40%.
- b) The building period T (in secs) may be approximated by the following formula:

$$T = C_t (h_n)^m \quad (2.5.8)$$

where,

h_n = Height of building in metres from foundation or from top of rigid basement. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storeys, when they are not so connected.

C_t and m are obtained from Table 2.5.8

Table 2.5.8 Values for coefficients to estimate approximate period

Structure type	C_t	m
Concrete moment-resisting frames	0.0466	0.9
Steel moment-resisting frames	0.0724	0.8
Eccentrically braced steel frame	0.0731	0.75
All other structural systems	0.0488	0.75

NOTE:

Consider moment resisting frames as frames which resist 100% of seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting under seismic forces.

- c) For masonry or concrete shear wall structures, the approximate fundamental period, T in secs may be determined as follows:

$$T = \frac{0.0062}{\sqrt{C_w}} h_n \quad (2.5.9)$$

where,

$$C_w = \frac{100}{A_B} \sum_{i=1}^x \left(\frac{h_n}{h_i} \right)^2 \left[\frac{A_i}{1 + 0.83 \left(\frac{h_i}{D_i} \right)^2} \right] \quad (2.5.10)$$

where,

A_B = area of base of structure

A_i = web area of shear wall "i"

D_i = length of shear wall "i"

h_i = height of shear wall "i"

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

2.5.9.3 SEISMIC WEIGHT

Seismic weight, W , is the total dead load of a building or a structure, including partition walls, and applicable portions of other imposed loads listed below:

- a) For live load up to and including 3 kN/m^2 , a minimum of 25% of the live load shall be applicable.
- b) For live load above 3 kN/m^2 , a minimum of 50% of the live load shall be applicable.
- c) Total weight (100%) of permanent heavy equipment or retained liquid or any imposed load sustained in nature shall be included.

Where the probable imposed loads (mass) at the time of earthquake are more correctly assessed, the designer may go for higher percentage of live load.

2.5.9.4 VERTICAL DISTRIBUTION OF LATERAL FORCES

In the absence of a more rigorous procedure, the total seismic lateral force at the base level, in other words the base shear V , shall be considered as the sum of lateral forces F_x induced at different floor levels, these forces may be calculated as:

$$F_x = V \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (2.5.11)$$

where,

F_x = part of base shear force induced at level x

w_i and w_x = part of the total effective seismic weight of the structure (W) assigned to level i or x

h_i and h_x = the height from the base to level i or x

$k = 1$ for structure period $\leq 0.5\text{s}$

$= 2$ for structure period $\geq 2.5\text{s}$

$=$ linear interpolation between 1 and 2 for other periods.

n = number of stories

2.5.9.5 STOREY SHEAR AND ITS HORIZONTAL DISTRIBUTION

The design storey shear V_x , at any storey x is the sum of the forces F_x in that storey and all other stories above it, given by Eq. 2.5.12:

$$V_x = \sum_{i=x}^n F_i \quad (2.5.12)$$

where, F_i = Portion of base shear induced at level i , as determined by Eq. 2.5.11

If the floor diaphragms can be considered to be infinitely rigid in the horizontal plane, the shear V_x shall be distributed to the various elements of the lateral force resisting system in proportion to their relative lateral stiffness. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

Allowance shall also be made for the increased shear arising due to horizontal torsional moment as specified in Sec 2.5.9.6

2.5.9.6 HORIZONTAL TORSIONAL MOMENTS :

Design shall accommodate increase in storey shear forces resulting from probable horizontal torsional moments on rigid floor diaphragms. Computation of such moments shall be as follows:

2.5.9.6.1 In-built Torsional Effects:

When there is in-built eccentricity between centre of mass and centre of rigidity (lateral resistance) at the floor levels, rigid diaphragms at each level will be subjected to torsional moment M_t .

2.5.9.6.2 Accidental Torsional Effects:

In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, accidental torsional effects need to be always considered. The accidental moment M_{ta} is determined assuming the storey mass to be displaced from the calculated centre of mass a distance equal to 5% of the building dimension at that level perpendicular to the direction of the force under consideration. The accidental torsional moment M_{tai} at level i is given as:

$$M_{tai} = e_{ai} F_i \quad (2.5.13)$$

where,

e_{ai} = accidental eccentricity of floor mass at level i applied in the same direction at all floors = $\pm 0.05L_i$

L_i = floor dimension perpendicular to the direction of seismic force considered.

Where torsional irregularity exists (Section 2.5.7.3.1) for Seismic Design Category C or D, the irregularity effects shall be accounted for by increasing the accidental torsion M_{ta} at each level by a torsional amplification factor, A_x as illustrated in Figure 2.5.6 determined from the following equation:

$$A_x = \left[\delta_{\max} / (1.2\delta_{\text{avg}}) \right]^2 \leq 3.0 \quad (2.5.14)$$

where,

$\delta_{\max}$ = Maximum displacement at level- x computed assuming $A_x=1$.

δ_{avg} = Average of the displacements at extreme points of the building at level- x computed assuming $A_x = 1$.

The accidental torsional moment need not be amplified for structures of light-frame construction. Also the torsional amplification factor (A_x) should not exceed 3.0.

2.5.9.6.3 Design for Torsional Effects

The torsional design moment at a given storey shall be equal to the accidental torsional moment M_{ta} plus the inbuilt torsional moment M_t (if any). Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass (for accidental torsion) need not be applied in both of the orthogonal directions at the same time, but shall be applied in only one direction that produces the greater effect.

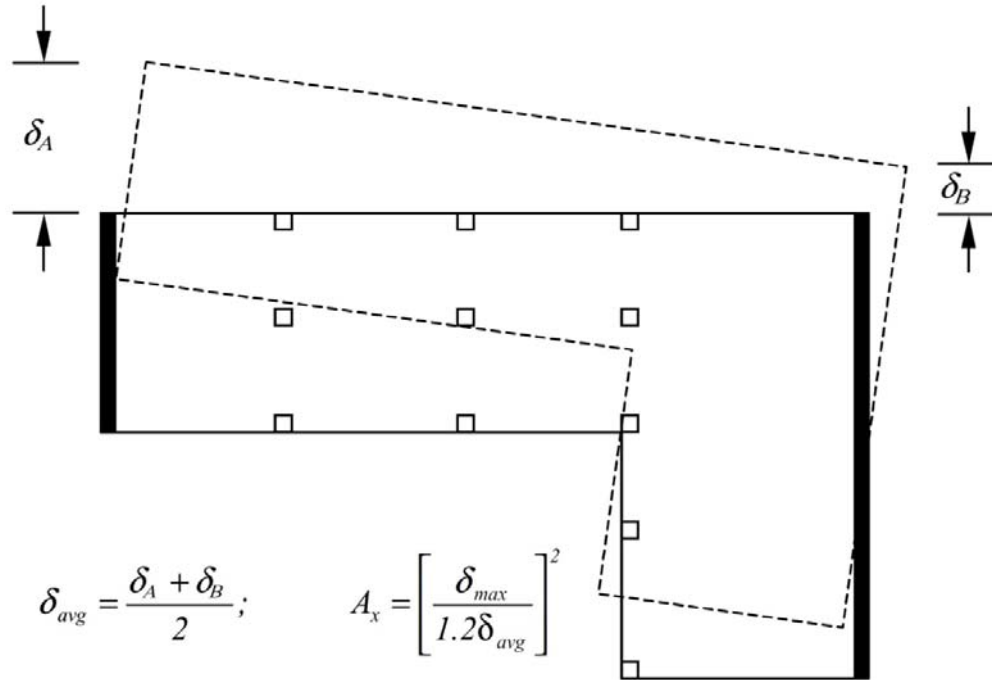


Figure 2.5.6 Torsional amplification factor A_x for plan irregularity.

2.5.9.7 DEFLECTION AND STOREY DRIFT :

The deflections (δ_x) of level x at the center of the mass shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \quad (2.5.15)$$

where,

C_d = Deflection amplification factor given in Table 2.5.7

δ_{xe} = Deflection determined by an elastic analysis

I = Importance factor defined in Table 2.5.5

The design storey drift at storey x shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration:

$$\Delta_x = \delta_x - \delta_{x-1} \quad (2.5.16)$$

2.5.9.8 OVERTURNING EFFECTS:

The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 2.5.9.4. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical force resisting elements in the same proportion as the distribution of the horizontal shears to those elements.

The overturning moments at level x , M_x , shall be determined as follows:

$$M_x = \sum_{i=x}^n F_i (h_i - h_x) \quad (2.5.17)$$

where,

F_i = the portion of the seismic base shear, V , induced at level i

h_i, h_x = the height from the base to level i or x .

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for three-fourths of the foundation overturning design moment, M_o , determined using above equation.

2.5.9.9 P-DELTA EFFECTS:

The P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered if the stability coefficient (θ) determined by the following equation is not more than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \quad (2.5.18)$$

Where,

P_x = the total vertical design load at and above level x ; where computing P_x , no individual load factor need exceed 1.0

Δ = the design story drift occurring simultaneously with V_x

V_x = the storey shear force acting between levels x and $x - 1$

h_{sx} = the story height below level x

C_d = the deflection amplification factor given in Table 2.5.7

The stability coefficient (θ) shall not exceed $\theta_{\max}$ determined as follows:

$$\theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25 \quad (2.5.19)$$

where β is the ratio of shear demand to shear capacity for the story between levels x and $x - 1$. This ratio is permitted to be conservatively taken as 1.0.

Where the stability coefficient (θ) is greater than 0.10 but less than or equal to $\theta_{\max}$, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $1.0/(1 - \theta)$.

Where θ is greater than $\theta_{\max}$, the structure is potentially unstable and shall be redesigned.

Where the P-delta effect is included in an automated analysis, Eq. 2.5.19 shall still be satisfied, however, the value of θ computed from Eq. 2.5.18 using the results of the P-delta analysis is permitted to be divided by $(1 + \theta)$ before checking Eq. 2.5.19.

2.5.10 DYNAMIC ANALYSIS METHODS

Dynamic analysis method involves applying principles of structural dynamics to compute the response of the structure to applied dynamic (earthquake) loads.

2.5.10.1 REQUIREMENT FOR DYNAMIC ANALYSIS

Dynamic analysis should be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

- a) Regular buildings with height greater than 40 m in Zones 2, 3, 4 and greater than 90 m in Zone 1.
- b) Irregular buildings (as defined in Section 2.5.7.3) with height greater than 12 m in Zones 2, 3, 4 and greater than 40 m in Zone 1.

For irregular buildings, smaller than 40 m in height in Zone 1, dynamic analysis, even though not mandatory, is recommended.

2.5.10.2 METHODS OF ANALYSIS

Dynamic analysis may be carried out through the following two methods:

- (i) Response Spectrum Analysis method is a linear elastic analysis method using modal analysis procedures, where the structure is subjected to spectral accelerations corresponding to a design acceleration response spectrum. The design earthquake ground motion in this case is represented by its response spectrum.
- (ii) Time History Analysis method is a numerical integration procedure where design ground motion time histories (acceleration record) are applied at the base of the structure. Time history analysis procedures can be two types: linear and non-linear.

2.5.11 RESPONSE SPECTRUM ANALYSIS (RSA)

A response spectrum analysis shall consist of the analysis of a linear mathematical model of the structure to determine the maximum accelerations, forces, and displacements resulting from the dynamic response to ground shaking represented by the design acceleration response spectrum (presented in Section 2.5.6.3). Response spectrum analysis is also called a modal analysis procedure because it considers different modes of vibration of the structure and combines effects of different modes.

2.5.11.1 MODELING (RSA)

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models are permitted to be constructed to represent each system. For irregular structures or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model should include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the

participation of the diaphragm in the structure's dynamic response. The structure shall be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections
2. The contribution of panel zone deformations to overall story drift shall be included for steel moment frame resisting systems.

2.5.11.2 NUMBER OF MODES (RSA)

An analysis shall be conducted using the masses and elastic stiffnesses of the seismic-force-resisting system to determine the natural modes of vibration for the structure including the period of each mode, the modal shape vector ϕ , the modal participation factor P and modal mass M . The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions.

2.5.11.3 MODAL STORY SHEARS AND MOMENTS (RSA)

For each mode, the story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces shall be computed. The peak lateral force F_{ik} induced at level i in mode k is given by:

$$F_{ik} = A_k \phi_{ik} P_k W_i \quad (2.5.20)$$

where,

A_k = Design horizontal spectral acceleration corresponding to period of vibration T_k of mode k , obtained from design response spectrum (Section 2.5.6.3)

ϕ_{ik} = Modal shape coefficient at level i in mode k

P_k = Modal participation factor of mode k

W_i = Weight of floor i .

2.5.11.4 STRUCTURE RESPONSE (RSA)

In the response spectrum analysis method, the base shear V_{rs} ; each of the story 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 (SRSS) of each of the modal values or by the complete quadratic combination (CQC) technique. The complete quadratic combination shall be used where closely spaced periods in the translational and torsional modes result in cross-correlation of the modes.

The distribution of horizontal shear shall be in accordance with the requirements of Section 2.5.9.5. It should be noted that amplification of accidental torsion as per Section 2.5.9.6 is not required where accidental torsional effects are included in the dynamic analysis model by offsetting the centre of mass in each story by the required amount.

A base shear, V , shall also be calculated using the equivalent static force procedure in Section 2.5.9. Where the base shear, V_{rs} , is less than 85% of V , all the forces but not the drifts obtained by response spectrum analysis shall be multiplied by the ratio $0.85V/V_{rs}$.

The displacements and drifts obtained by response spectrum analysis shall be multiplied by C_d/I to obtain design displacements and drifts, as done in equivalent static analysis procedure (Section 2.5.9.7).

The P-delta effects shall be determined in accordance with Section 2.5.9.9.

2.5.12 LINEAR TIME HISTORY ANALYSIS (LTHA)

A linear time history analysis (LTHA) shall consist of an analysis of a linear mathematical model of the structure to determine its response, through direct numerical integration of the differential equations of motion, to a number of ground motion acceleration time histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the provisions of this section. For the purposes of analysis, the structure shall be permitted to be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. The acceleration time history (ground motion) is applied at the base of the structure. The advantage of this procedure is that the time dependent behavior of the structural response is obtained.

2.5.12.1 MODELING (LTHA)

Mathematical models shall conform to the requirements of modeling described in Section 2.5.11.1.

2.5.12.2 GROUND MOTION (LTHA)

At least three appropriate ground motions (acceleration time history) shall be used in the analysis. Ground motion shall conform to the requirements of this section.

Two-dimensional analysis: Where two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration time history selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of appropriate ground motion records are not available, appropriate simulated ground motion time histories shall be used to make up the total number required. The ground motions shall be scaled such that for each period between $0.2T$ and $1.5T$ (where T is the natural period of the structure in the fundamental mode for the direction considered) the average of the five-percent-damped response spectra for the each acceleration time history is not less than the corresponding ordinate of the design acceleration response spectrum, determined in accordance with Section 2.5.6.3 .

Three-dimensional analysis: Where three-dimensional analysis is performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration time histories (in two orthogonal horizontal directions) that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, an SRSS spectrum shall be constructed by taking the square root of the sum of the squares of the five-percent-damped response spectra for the components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between $0.2T$ and $1.5T$ (where T is the natural period of the fundamental mode of the structure) the average of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the design response spectrum, determined in accordance with Section 2.5.6.3.

2.5.12.3 STRUCTURE RESPONSE (LTHA)

For each scaled acceleration time history, the maximum values of base shear and other structure response quantities shall be obtained from the time history analysis. For three dimensional analysis, orthogonal pair of scaled motions are applied simultaneously. A base shear, V , shall also be calculated using the equivalent static force procedure described in Section 2.5.9.1. Where the maximum base shear, V_{th} computed by linear time history analysis, is less than V , all response quantities (storey shear, moments, drifts, floor deflections, member forces etc) obtained by time history analysis shall be increased by multiplying with the ratio V/V_{th} . If number of earthquake records (or pairs) used in the analysis is less than seven, the maximum structural response obtained corresponding to different earthquake records shall be considered as the design value. If the number is at least seven, then the average of maximum structural responses for different earthquake records shall be considered as the design value.

The displacements and drifts obtained as mentioned above shall be multiplied by C_d/I to obtain design displacements and drifts, as done in equivalent static analysis procedure (Section 2.5.9.7).

2.5.13 NON-LINEAR TIME HISTORY ANALYSIS (NTHA)

Nonlinear time history analysis (NTHA) shall consist of analysis of a mathematical model of the structure which incorporates the nonlinear hysteretic behavior of the structure's components to determine its response, through methods of numerical integration, to ground acceleration time histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the requirements of this section. For the purposes of analysis, the structure shall be permitted to be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. The acceleration time history (ground motion) is applied at the base of the structure. The advantage of this procedure is that actual time dependent behavior of the structural response considering inelastic deformations in the structure can be obtained.

2.5.13.1 MODELING (NTHA)

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass throughout the structure. The hysteretic behavior of elements shall be modeled consistent with suitable laboratory test data and shall account for all significant yielding, strength degradation, stiffness degradation, and hysteretic pinching indicated by such test data. Strength of elements shall be based on expected values considering material overstrength, strain hardening, and hysteretic strength degradation. As a minimum, a bilinear force–deformation relationship should be used at the element level. In reinforced concrete and masonry buildings, the elastic stiffness should correspond to that of cracked sections. Linear properties, consistent with the provisions of Chapter 5 shall be permitted to be used for those elements demonstrated by the analysis to remain within their linear range of response. The structure shall be assumed to have a fixed base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness and load carrying characteristics of the foundations consistent with site-specific soils data and rational principles of engineering mechanics.

For regular structures with independent orthogonal seismic-force-resisting systems, independent two dimensional models shall be permitted to be constructed to represent each system. For structures having plan irregularity or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the structure shall be used.

Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model shall include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

2.5.13.2 GROUND MOTION (NTHA)

The actual time-dependent inelastic deformation of the structure is modeled. For inelastic analysis method, the real design acceleration response spectrum (Section 2.5.6.3) is obtained using Eq.2.5.4 with $R=1$ and $I=1$. The real design acceleration response spectrum is the true representation of the expected ground motion (design basis earthquake) including local soil effects and corresponds to a peak ground acceleration (PGA) value of $2/3 \cdot Z \cdot S$.

At least three appropriate acceleration time histories shall be used in the analysis. Ground motion shall conform to the requirements of this section.

Two-dimensional analysis: Where two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration time history selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of appropriate ground motion records are not available, appropriate simulated ground motion time histories shall be used to make up the total number required. The ground motions shall be scaled such that for each period between $0.2T$ and $1.5T$ (where T is the natural period of the structure in the fundamental mode for the direction considered) the average of the five-percent-damped response spectra for each acceleration time history is not less than the corresponding ordinate of the real design acceleration response spectrum, as defined here.

Three-dimensional analysis: Where three-dimensional analysis is performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration time histories (in two orthogonal horizontal directions) that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, an SRSS spectrum shall be constructed by taking the square root of the sum of the squares of the five-percent-damped response spectra for the components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between $0.2T$ and $1.5T$ (where T is the natural period of the fundamental mode of the structure) the average of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the real design acceleration response spectrum.

2.5.13.3 STRUCTURE RESPONSE (NTHA)

For each scaled acceleration time history, the maximum values of base shear and other structure response quantities shall be obtained from the nonlinear time history analysis. For three dimensional analysis, orthogonal pair of scaled motions are applied simultaneously. If number of earthquake records (or pairs) used in the analysis is less than seven, the maximum structural response obtained corresponding to different earthquake records shall be considered as the design value. If the number is at least seven, then

the average of maximum structural responses for different earthquake records shall be considered as the design value. Since real expected earthquake motion input and model incorporating real nonlinear behavior of the structure is used, the results as obtained are directly used (no scaling as in LTHA or RSA is required) for interpretation and design.

2.5.13.4 STRUCTURE MEMBER DESIGN (NTHA)

The adequacy of individual members and their connections to withstand the design deformations predicted by the analyses shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. Member deformation shall not exceed two thirds of the smaller of: the value that results in loss of ability to carry gravity loads or the value at which member strength has deteriorated to less than 67% of peak strength.

2.5.13.5 DESIGN REVIEW (NTHA)

Special care and expertise is needed in the use of nonlinear dynamic analysis based design. Checking of the design by competent third party is recommended. A review of the design of the seismic-force-resisting system and the supporting structural analyses shall be performed by an independent team consisting of design professionals with experience in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under extreme cyclic loads. The design review shall include the following: (i) Review of development of ground motion time histories (ii) Review of acceptance criteria (including laboratory test data) used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and deformation demands (iii) Review of structural design.

2.5.14 NON-LINEAR STATIC ANALYSIS (NSA)

Nonlinear static analysis (NSA), also popularly known as pushover analysis, is a simplified method of directly evaluating nonlinear response of structures to strong earthquake ground shaking. It is an alternative to the more complex nonlinear time history analysis (NTHA). The building is subjected to monotonically increasing static horizontal loads under constant gravity load.

2.5.14.1 MODELING (NSA)

A mathematical model of the structure shall be constructed to represent the spatial distribution of mass and stiffness of the structural system considering the effects of element nonlinearity for deformation levels that exceed the proportional limit. P-Delta effects shall also be included in the analysis.

For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models may be used to represent each system. For structures having plan irregularities or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three degrees of freedom for each level of the structure, consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis, shall be used. Where the diaphragms are not rigid compared to the vertical elements of the seismic-force-resisting system, the model should include representation of the diaphragm flexibility.

Unless analysis indicates that a element remains elastic, a nonlinear force deformation model shall be used to represent the stiffness of the element before onset of yield, the yield strength, and the stiffness properties of the element after yield at various levels of deformation. Strengths of elements shall not exceed expected values considering material overstrength and strain hardening. The properties of

elements and components after yielding shall account for strength and stiffness degradation due to softening, buckling, or fracture as indicated by principles of mechanics or test data.

A control point shall be selected for the model. For normal buildings, the control point shall be at the center of mass of the highest level (roof) of the structure.

2.5.14.2 ANALYSIS PROCEDURE (NSA)

The lateral forces shall be applied at the center of mass of each level and shall be proportional to the distribution obtained from a modal analysis for the fundamental mode of response in the direction under consideration. The lateral loads shall be increased incrementally in a monotonic manner.

At the j -th increment of lateral loading, the total lateral force applied to the model shall be characterized by the term V_j . The incremental increases in applied lateral force should be in steps that are sufficiently small to permit significant changes in individual element behavior (such as yielding, buckling or failure) to be detected. The first increment in lateral loading shall result in linear elastic behavior. At each loading step, the total applied lateral force, V_j , the lateral displacement of the control point, δ_j , and the forces and deformations in each element shall be recorded. The analysis shall be continued until the displacement of the control point is at least 150% of the target displacement determined in accordance with Sec.2.5.14.3. The structure shall be designed so that the total applied lateral force does not decrease in any load increment for control point displacements less than or equal to 125 percent of the target displacement.

2.5.14.3 EFFECTIVE PERIOD AND TARGET DISPLACEMENT (NSA)

A bilinear curve shall be fitted to the capacity curve, such that the first segment of the bilinear curve coincides with the capacity curve at 60% of the effective yield strength, the second segment coincides with the capacity curve at the target displacement, and the area under the bilinear curve equals the area under the capacity curve, between the origin and the target displacement. The effective yield strength, V_y , corresponds to the total applied lateral force at the intersection of the two line segments. The effective yield displacement, δ_y , corresponds to the control point displacement at the intersection of the two line segments. The effective fundamental period, T_e , of the structure in the direction under consideration shall be determined using Eq. 2.5.21 as follows:

$$T_e = T_1 \sqrt{\frac{V_1/\delta_1}{V_y/\delta_y}} \quad (2.5.21)$$

where V_1 , δ_1 , and T_1 are determined for the first increment of lateral load.

The target displacement of the control point, δ_T , shall be determined as follows:

$$\delta_T = C_0 C_1 S_a \left(\frac{T_e}{2\pi} \right)^2 g \quad (2.5.22)$$

where the spectral acceleration, S_a , is determined at the effective fundamental period, T_e , using Eq.(2.5.4), g is the acceleration due to gravity.

The coefficient C_0 shall be calculated as:

$$C_0 = \frac{\sum_{i=1}^n w_i \phi_i}{\sum_{i=1}^n w_i \phi_i^2} \quad (2.5.23)$$

where:

w_i = the portion of the seismic weight, W , at level i , and

ϕ_i = the amplitude of the shape vector at level i .

Where the effective fundamental period, T_e , is greater than T_c (defined in Sec. 2.5.6.3), the coefficient C_1 shall be taken as 1.0. Otherwise, the value of the coefficient C_1 shall be calculated as follows:

$$C_1 = \frac{1}{R_d} \left(1 + \frac{(R_d - 1)T_s}{T_e} \right) \quad (2.5.24)$$

where R_d is given as follows:

$$R_d = \frac{S_a}{V_y/W} \quad (2.5.25)$$

2.5.14.4 STRUCTURE MEMBER DESIGN (NSA)

For each nonlinear static analysis the design response parameters, including the individual member forces and member deformations shall be taken as the values obtained from the analysis at the step at which the target displacement is reached.

The adequacy of individual members and their connections to withstand the member forces and member deformations shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. The deformation of a member supporting gravity loads shall not exceed (i) two-thirds of the deformation that results in loss of ability to support gravity loads, and (ii) two-thirds of the deformation at which the member strength has deteriorated to less than 70% of the peak strength of the component model. The deformation of a member not required for gravity load support shall not exceed two-thirds of the value at which member strength has deteriorated to less than 70% of the peak strength of the component model.

2.5.14.5 DESIGN REVIEW (NSA)

Checking of the design by competent third party is recommended. An independent team composed of at least two members with experience in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under earthquake loading, shall perform a review of the design of the seismic force resisting system and the supporting structural analyses. The design review shall include (i) review of any site-specific seismic criteria (if developed) employed in the analysis (ii) review of the determination of the target displacement and effective yield strength of the structure (iii) review of adequacy of structural elements and systems to withstand the calculated force and deformation demands, together with laboratory and other data (iv) review of structural design.

2.5.15 EARTHQUAKE LOAD COMBINATIONS

2.5.15.1 HORIZONTAL EARTHQUAKE LOADING

The directions of application of seismic forces for design shall be those which will produce the most critical load effects. Earthquake forces act in both principal directions of the building simultaneously. In order to account for that,

- (a) For structures of Seismic Design Category B, the design seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected
- (b) Structures of Seismic Design Category C and D shall, as a minimum, conform to the requirements of (a) for Seismic Design Category B and in addition the requirements of this section. The structure shall be designed for 100% of the seismic forces in one principal direction combined with 30% of the seismic forces in the orthogonal direction. Possible combinations are:

“ $\pm 100\%$ in x-direction $\pm 30\%$ in y-direction” or

“ $\pm 30\%$ in x-direction $\pm 100\%$ in y-direction”

The combination which produces most unfavourable effect for the particular action effect shall be considered. This approach may be applied to equivalent static analysis, response spectrum analysis and linear time history analysis procedure.

- (c) Where three-dimensional analysis of a spatial structure model is performed as in 3D time history analysis, simultaneous application of accelerations in two directions shall be considered where the ground motions shall satisfy the conditions stated in Sections 2.5.12.2 or 2.5.13.2.

2.5.15.2 VERTICAL EARTHQUAKE LOADING

The maximum vertical ground acceleration shall be taken as 50% of the expected horizontal peak ground acceleration (PGA). The vertical seismic load effect E_v may be determined as:

$$E_v = 0.5(a_h)D \quad (2.5.26)$$

where,

a_h = expected horizontal peak ground acceleration (in g) for design = $(2/3)ZS$

D = effect of dead load

2.5.15.3 COMBINATION OF EARTHQUAKE LOADING WITH OTHER LOADINGS

When earthquake effect is included in the analysis and design of a building or structure, the provisions set forth in section 2.7 shall be followed to combine earthquake load effects with other loading effects to obtain design forces etc.

2.5.16 DRIFT AND DEFORMATION

2.5.16.1 STOREY DRIFT LIMIT

The design storey drift (Δ) of each storey, as determined in Sections 2.5.9 (equivalent static analysis), 2.5.11 (response spectrum analysis) or 2.5.12 (linear time history analysis) shall not exceed the allowable storey drift (Δ_a) as obtained from Table 2.5.9 for any story.

For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C or D having torsional irregularity, the design storey drift, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the storey under consideration. For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, the allowable storey drift for such linear elastic analysis procedures shall not exceed Δ_a/ρ where ρ is termed as a structural redundancy factor. The value of redundancy factor ρ may be considered as 1.0 with the exception of structures with very low level of redundancy where ρ may be considered as 1.3.

For nonlinear time history analysis (NTHA), the storey drift obtained (Section 2.5.13) shall not exceed 1.25 times the storey drift limit specified above for linear elastic analysis procedures.

Table 2.5.9 Allowable Storey Drift Limit (Δ_a)

Structure	Occupancy Category		
	I and II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	0.025 h_{sx}	0.020 h_{sx}	0.015 h_{sx}
Masonry cantilever shear wall structures	0.010 h_{sx}	0.010 h_{sx}	0.010 h_{sx}
Other masonry shear wall structures	0.007 h_{sx}	0.007 h_{sx}	0.007 h_{sx}
All other structures	0.020 h_{sx}	0.015 h_{sx}	0.010 h_{sx}
NOTES:			
1. h_{sx} is the story height below Level x.			
2. There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts.			
3. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.			
4. Occupancy categories are defined in Table 1.2.1			

2.5.16.2 DIAPHRAGM DEFLECTION

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 that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

2.5.16.3 SEPARATION BETWEEN ADJACENT STRUCTURES

Buildings shall be protected from earthquake-induced pounding from adjacent structures or between structurally independent units of the same building maintaining safe distance between such structures as follows:

- (i) for buildings, or structurally independent units, that do not belong to the same property, the distance from the property line to the potential points of impact shall not be less than the computed maximum horizontal displacement (Section 2.5.9.7) of the building at the corresponding level.
- (ii) for buildings, or structurally independent units, belonging to the same property, if the distance between them is not less than the square root of the sum- of the squares (SRSS) of the computed maximum horizontal displacements (Section 2.5.9.7) of the two buildings or units at the corresponding level.
- (iii) If the floor elevations of the building or independent unit under design are the same as those of the adjacent building or unit, the above referred minimum distance may be reduced by a factor of 0.7

2.5.16.4 SPECIAL DEFORMATION REQUIREMENT FOR SEISMIC DESIGN CATEGORY D

For structures assigned to Seismic Design Category D, every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from displacement to the design story drift (Δ) as determined in accordance with Section 2.5.9.7. Even where elements of the structure are not intended to resist seismic forces, their protection may be important. Where determining the moments and shears induced in components that are not included in the seismic force-resisting system in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used.

2.5.17 SEISMIC DESIGN FOR NONSTRUCTURAL COMPONENTS

This section establishes minimum design criteria for nonstructural components that are permanently attached to structures and for their supports and attachments.

The following components are exempt from the requirements of this section.

1. Architectural components in Seismic Design Category B, other than parapets supported by bearing walls or shear walls, where the component importance factor, I_c , is equal to 1.0.
2. Mechanical and electrical components in Seismic Design Category B.
3. Mechanical and electrical components in Seismic Design Category C where the importance factor, I_c , is equal to 1.0.
4. Mechanical and electrical components in Seismic Design Category D where the component importance factor, I_c , is equal to 1.0 and either (a) flexible connections between the components and associated ductwork, piping, and conduit are provided, or (b) components are mounted at 1.2 m or less above a floor level and weigh 1780 N or less.
5. Mechanical and electrical components in Seismic Design Category C or D where the component importance factor, I_c , is equal to 1.0 and (a) flexible connections between the components and associated ductwork, piping, and conduit are provided, and (b) the components weigh 95 N or less or, for distribution systems, which weigh 7 N/m or less.

Where the individual weight of supported components and nonbuilding structures with periods greater than 0.06 seconds exceeds 25% of the total seismic weight W , the structure shall be designed considering interaction effects between the structure and the supported components.

Testing shall be permitted to be used in lieu of analysis methods outlined in this chapter to determine the seismic capacity of components and their supports and attachments.

2.5.17.1 COMPONENT IMPORTANCE FACTOR

All components shall be assigned a component importance factor. The component importance factor, I_c , shall be taken as 1.5 if any of the following conditions apply:

1. The component is required to function after an earthquake,
2. The component contains hazardous materials, or
3. The component is in or attached to a occupancy category IV building and it is needed for continued operation of the facility.

All other components shall be assigned a component importance factor, I_c , equal to 1.0.

2.5.17.2 COMPONENT FORCE TRANSFER

Components shall be attached such that the component forces are transferred to the structure. Component attachments that are intended to resist seismic forces shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be verified. Local elements of the supporting structure shall be designed for the component forces where such forces control the design of the elements or their connections. In this instance, the component forces shall be those determined in Section 2.5.17.3, except that modifications to F_p and R_p due to anchorage conditions need not be considered. The design documents shall include sufficient information concerning the attachments to verify compliance with the requirements of these Provisions.

2.5.17.3 SEISMIC DESIGN FORCE

The seismic design force, F_c , applied in the horizontal direction shall be centered at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined as follows:

$$F_c = \frac{\alpha_c a_h W_c I_c}{R_c} \left(1 + 2 \frac{z}{h} \right) \quad (2.5.27)$$

Where,

$$0.75W_c I_c \leq F_c \leq 1.5W_c I_c$$

α_c = component amplification factor which varies from 1.0 to 2.5 (Table 2.5.10 or Table 2.5.11).

a_h = expected horizontal peak ground acceleration (in g) for design = $2/3 * Z * S$

W_c = weight of component

R_c = component response reduction factor which varies from 1.0 to 12.0 (Table 2.5.10 or Table 2.5.11)

z = height above the base of the point of attachment of the component, but z shall not be taken less than 0 and the value of z/h need not exceed 1.0

h = roof height of structure above the base

The force F_c shall be independently applied in at least two orthogonal horizontal directions in combination with service loads associated with the component. In addition, the component shall also be designed for a concurrent vertical force of $\pm 0.5a_h W_c$.

Where nonseismic loads on nonstructural components exceed F_c , such loads shall govern the strength design, but the seismic detailing requirements and limitations shall apply.

2.5.17.4 SEISMIC RELATIVE DISPLACEMENTS

The relative seismic displacement, D_c , for two connection points on the same structure A, one at a height h_x and other at height h_y , for use in component design shall be determined as follows:

$$D_c = \delta_{xA} - \delta_{yA} \quad (2.5.28)$$

D_c shall not exceed $D_{c \max}$ given by:

$$D_{c \max} = \frac{(h_x - h_y) \Delta_{aA}}{h_{sx}} \quad (2.5.29)$$

where,

δ_{xA} = Deflection at level x of structure A

δ_{yA} = Deflection at level y of structure A

Δ_{aA} = Allowable story drift for structure A

h_x = Height (above base) of level x to which upper connection point is attached.

h_y = Height (above base) of level y to which lower connection point is attached.

h_{sx} = Story height used in the definition of the allowable drift Δ_a

For two connection points on separate structures, A and B, or separate structural systems, one at level x and the other at level y, D_c shall be determined as follows:

$$D_c = |\delta_{xA}| + |\delta_{yB}| \quad (2.5.30)$$

D_c shall not exceed $D_{c \max}$ given by:

$$D_{c \max} = \frac{X \Delta_{aA}}{h_{sx}} + \frac{Y \Delta_{aB}}{h_{sy}} \quad (2.5.31)$$

Where,

δ_{yB} = Deflection at level y of structure B

Δ_{aB} = Allowable story drift for structure B

The effects of relative seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

Table 2.5.10 Coefficients α_c and R_c for Architectural Components

Architectural Component or Element	α_c^a	R_c
Interior Nonstructural Walls and Partitions		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever Elements (Unbraced or braced to structural frame below its center of mass)		
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks where laterally braced or supported by the structural frame	2.5	2.5
Cantilever Elements (Braced to structural frame above its center of mass) Parapets		
Chimneys and Stacks	1.0	2.5
Exterior Nonstructural Walls	1.0	2.5
Exterior Nonstructural Wall Elements and Connections		
Wall Element	1.0	2.5
Body of wall panel connections	1.0	2.5
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Storage cabinets and laboratory equipment	1.0	2.5
Access Floors		
Special access floors	1.0	2.5
All other	1.0	1.5
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other Flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5

^a A lower value for α_c is permitted where justified by detailed dynamic analysis. The value for α_c shall not be less than 1.0. The value of α_c equal to 1.0 is for rigid components and rigidly attached components. The value of α_c equal to 2.5 is for flexible components and flexibly attached components.

Table 2.5.11 Coefficients α_c and R_c for Mechanical and Electrical Components

Mechanical and Electrical Components	α_c^a	R_c
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal	2.5	6.0
Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts	1.0	2.5
Skirt-supported pressure vessels	1.0	2.5
Elevator and escalator components.	2.5	2.5
Generators, batteries, inverters, motors, transformers, and other electrical components	1.0	2.5
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other	2.5	6.0
Communication equipment, computers, instrumentation, and controls.	1.0	2.5
Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced below	2.5	3.0
Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced above	1.0	2.5
Lighting fixtures.	1.0	1.5
Other mechanical or electrical components.	1.0	1.5
VIBRATION ISOLATED COMPONENTS AND SYSTEMS^b		
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter	2.5	2.5
Spring isolated components and systems and vibration isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient	2.5	2.0
Internally isolated components and systems.	2.5	2.0
Suspended vibration isolated equipment including in-line duct devices and suspended	2.5	2.5
DISTRIBUTION SYSTEMS		
Piping in accordance with ASME B31, including in-line components with joints made	2.5	12.0
Piping in accordance with ASME B31, including in-line components, constructed of high or limited deformability materials, with joints made by threading, bonding,	2.5	6.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2.5	9.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading,	2.5	4.5
Piping and tubing constructed of low-deformability materials, such as cast iron, glass,	2.5	3.0
Ductwork, including in-line components, constructed of high-deformability materials,	2.5	9.0
Ductwork, including in-line components, constructed of high- or limited-deformability materials with joints made by means other than welding or brazing.	2.5	6.0
Ductwork, including in-line components, constructed of low-deformability materials,	2.5	3.0
Electrical conduit, bus ducts, rigidly mounted cable trays, and plumbing.	1.0	2.5
Manufacturing or process conveyors (nonpersonnel).	2.5	3.0
Suspended cable trays.	2.5	6.0

^a A lower value for α_c is permitted where justified by detailed dynamic analysis. The value for α_c shall not be less than 1.0. The value of α_c equal to 1.0 is for rigid components and rigidly attached components. The value of α_c equal to 2.5 is for flexible components and flexibly attached components.

^b Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_c$ if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 6 mm. If the nominal clearance specified on the construction documents is not greater than 6 mm, the design force may be taken as F_c .

2.5.18 DESIGN FOR SEISMICALLY ISOLATED BUILDINGS

Buildings that use special seismic isolation systems for protection against earthquakes shall be called seismically isolated or base isolated buildings. Seismically isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of provisions presented in this section.

2.5.18.1 GENERAL REQUIREMENTS FOR ISOLATION SYSTEM

The isolation system to be used in seismically isolated structures shall satisfy the following requirements:

1. Design of isolation system shall consider variations in seismic isolator material properties over the projected life of structure including changes due to ageing, contamination, exposure to moisture, loadings, temperature, creep, fatigue, etc.
2. Isolated structures shall resist design wind loads at all levels above the isolation interface. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.
3. The fire resistance rating for the isolation system shall be consistent with the requirements of columns, walls, or other such elements in the same area of the structure.
4. The isolation system shall be configured to produce a lateral restoring force such that the lateral force at the total design displacement is at least 0.025W greater than the lateral force at 50% of the total design displacement.
5. The isolation system shall not be configured to include a displacement restraint that limits lateral displacement due to the maximum considered earthquake to less than the total maximum displacement unless it is demonstrated by analysis that such engagement of restraint does not result in unsatisfactory performance of the structure.
6. Each element of the isolation system shall be designed to be stable under the design vertical load when subjected to a horizontal displacement equal to the total maximum displacement.
7. The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum considered earthquake and the vertical restoring force shall be based on the seismic weight above the isolation interface.
8. Local uplift of individual units of isolation system is permitted if the resulting deflections do not cause overstress or instability of the isolator units or other elements of the structure.
9. Access for inspection and replacement of all components of the isolation system shall be provided.
10. The designer of the isolation system shall establish a quality control testing program for isolator units. Each isolator unit before installation shall be tested under specified vertical and horizontal loads.

11. After completion of construction, a design professional shall complete a final series of inspections or observations of structure separation areas and components that cross the isolation interface. Such inspections and observations shall confirm that existing conditions allow free and unhindered displacement of the structure to maximum design levels and that all components that cross the isolation interface as installed are able to accommodate the stipulated displacements.
12. The designer of the isolation system shall establish a periodic monitoring, inspection, and maintenance program for such system.
13. Remodeling, repair, or retrofitting at the isolation interface, including that of components that cross the isolation interface, shall be performed under the direction of a design professional experienced in seismic isolation systems.

2.5.18.2 EQUIVALENT STATIC ANALYSIS

The equivalent static analysis procedure is permitted to be used for design of a seismically isolated structure provided that:

1. The structure is located on Site Class SA, SB, SC, SD or SE site;
2. The structure above the isolation interface is not more than four stories or 20 m in height
4. The effective period of the isolated structure at the maximum displacement, T_M , is less than or equal to 3.0 sec.
5. The effective period of the isolated structure at the design displacement, T_D , is greater than three times the elastic, fixed-base period of the structure above the isolation system as determined in Sec. 2.5.9.2
6. The structure above the isolation system is of regular configuration; and
7. The isolation system meets all of the following criteria:
 - a. The effective stiffness of the isolation system at the design displacement is greater than one third of the effective stiffness at 20% of the design displacement,
 - b. The isolation system is capable of producing a restoring force as specified in Sec. 2.5.18.1,
 - c. The isolation system does not limit maximum considered earthquake displacement to less than the total maximum displacement.

Where the equivalent lateral force procedure is used to design seismically isolated structures, the requirements of this section shall apply.

2.5.18.2.1 Displacement of Isolation System

The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements that act in the direction of each of the main horizontal axes of the structure and such displacements shall be calculated as follows:

$$D_D = \frac{S_a g}{4\pi^2} \left(\frac{T_D^2}{B_D} \right) \quad (2.5.32)$$

where,

S_a = Design spectral acceleration (in units of g), calculated using Eq.(2.5.4) for period T_D and assuming $R=1$, $I=1$, $\eta=1$ (Section 2.5.6.3) for the design basis earthquake (DBE).

g = acceleration due to gravity

B_D = damping coefficient related to the effective damping β_D of the isolation system at the design displacement, as set forth in Table 2.5.12.

T_D = effective period of seismically isolated structure at the design displacement in the direction under consideration, as prescribed by Eq.2.5.33:

$$T_D = 2\pi \sqrt{\frac{W}{k_{Dmin} g}} \quad (2.5.33)$$

where,

W = seismic weight above the isolation interface

k_{Dmin} = minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

Table 2.5.12 Damping Coefficient, B_D or B_M

Effective Damping, β_D or β_M (%) ^{a,b}	B_D or B_M
≤ 2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
≥ 50	2.0

^a

The damping coefficient shall be based on the effective damping of the isolation system

^b

The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

The maximum displacement of the isolation system, D_M , in the most critical direction of horizontal response shall be calculated in accordance with the following formula:

$$D_M = \frac{S_{aM} g}{4\pi^2} \left(\frac{T_M^2}{B_M} \right) \quad (2.5.34)$$

where:

S_{aM} = Maximum spectral acceleration (in units of g), calculated using Eq.(2.5.4) for period T_D and assuming $R=1$, $I=1$, $\eta=1$ (Section 2.5.6.3) for the maximum considered earthquake (MCE).

B_M = numerical coefficient related to the effective damping β_M of the isolation system at the maximum displacement, as set forth in Table 2.5.12

T_M = effective period of seismic-isolated structure at the maximum displacement in the direction under consideration as prescribed by:

$$T_M = 2\pi \sqrt{\frac{W}{k_{M \min} g}} \quad (2.5.35)$$

where,

$k_{M \min}$ = minimum effective stiffness of the isolation system at the maximum displacement in the horizontal direction under consideration.

The total design displacement, D_{TD} , and the total maximum displacement, D_{TM} , of elements of the isolation system shall include additional displacement due to inherent and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of eccentric mass.

2.5.18.2.2 Lateral Seismic forces

The structure above the isolation system shall be designed and constructed to withstand a minimum lateral force, V_s , using all of the appropriate provisions for a non-isolated structure. The importance factor for all isolated structures shall be considered as 1.0, also the response reduction factor R_I considered here (for computing design seismic forces) is in the range of 1.0 to 2.0. V_s shall be determined in accordance with Eq. 2.5.36 as follows:

$$V_s = \frac{k_{D \max} D_D}{R_I} \quad (2.5.36)$$

where:

$k_{D \max}$ = maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

D_D = design displacement at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.5.32.

R_I = response reduction factor related to the type of seismic-force-resisting system above the isolation system. R_I shall be based on the type of seismic-force-resisting system used for the structure above the isolation system and shall be taken as the lesser of $3/8 \cdot R$ (Table 2.5.7) or 2.0, but need not be taken less than 1.0.

In no case shall V_s be taken less than the following:

1. The lateral force required by Section 2.5.9 for a fixed-base structure of the same weight, W , and a period equal to the isolated period, T_D ;
2. The base shear corresponding to the factored design wind load; and
3. The lateral force required to fully activate the isolation system (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the break-away friction level of a sliding system) multiplied by 1.5.

The isolation system, the foundation, and all structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral force, V_b , using all of the appropriate provisions for a non-isolated structure. V_b shall be determined in accordance with Eq. 2.5.37 as follows:

$$V_b = k_{Dmax} D_D \quad (2.5.37)$$

In all cases, V_b shall not be taken less than the maximum force in the isolation system at any displacement up to and including the design displacement.

2.5.18.2.3 Vertical Distribution of Lateral Forces

The total lateral force shall be distributed over the height of the structure above the isolation interface in accordance with Eq. 2.5.38 as follows:

$$F_x = V_s \frac{w_x h_x}{\sum_{i=1}^n w_i h_i} \quad (2.5.38)$$

where:

V_s = total seismic lateral design force on elements above the isolation system.

h_i, h_x = height above the base, to Level i or Level x , respectively.

w_i, w_x = portion of W that is located at or assigned to Level i or Level x , respectively.

At each Level x the force, F_x , shall be applied over the area of the structure in accordance with the distribution of mass at the level. Stresses in each structural element shall be determined by applying to an analytical model the lateral forces, F_x , at all levels above the base.

2.5.18.2.4 Storey Drift

The storey drift shall be calculated as in Section 2.5.9.7 except that C_d for the isolated structure shall be taken equal to R_i and importance factor equal to 1.0. The maximum storey drift of the structure above the isolation system shall not exceed $0.015h_{sx}$.

2.5.18.3 DYNAMIC ANALYSIS

Response spectrum analysis may be conducted if the behavior of the isolation system can be considered as equivalent linear. Otherwise, non-linear time history analysis shall be used where the true non-linear behaviour of the isolation system can be modeled. The mathematical models of the isolated structure including the isolation system shall be along guidelines given in Sections 2.5.11.1 and 2.5.13.1, and other requirements given in Section 2.5.18.

The isolation system shall be modeled using deformational characteristics developed and verified by testing. The structure model shall account for: (i) spatial distribution of isolator units; (ii) consideration of translation in both horizontal directions, and torsion of the structure above the isolation interface considering the most disadvantageous location of eccentric mass; (iii) overturning/uplift forces on individual isolator units; and (iv) effects of vertical load, bilateral load, and the rate of loading if the force-deflection properties of the isolation system are dependent on such attributes.

A linear elastic model of the isolated structure (above isolation system) may be used provided that: (i) stiffness properties assumed for the nonlinear components of the isolation system are based on the maximum effective stiffness of the isolation system, and (ii) all elements of the seismic-force-resisting system of the structure above the isolation system behave linearly.

2.5.18.3.1 Response Spectrum Analysis

Response spectrum analysis shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping values for higher modes shall be selected consistent with those that would be appropriate for response spectrum analysis of the structure above the isolation system assuming a fixed base.

Response spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the ground motion in the critical direction and 30 percent of the ground motion in the perpendicular, horizontal direction. The design basis earthquake shall be used for the design displacement, while the maximum considered earthquake shall be used for the maximum displacement. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements.

For the design displacement, structures that do not require site-specific ground motion evaluation, shall be analyzed using the design acceleration response spectrum in accordance with Section 2.5.6.3. The maximum design spectrum to be used for the maximum considered earthquake shall not be less than 1.5 times the design acceleration response spectrum.

The response spectrum procedure is based on an equivalent linear model, where the effective stiffness and effective damping is a function of the displacement, this formulation is thus an iterative process. The effective stiffness must be estimated, based on assumed displacement, and then adjusted till obtained displacement agree with assumed displacement.

The design shear at any story shall not be less than the story shear resulting from application of the story forces calculated using Eq. 2.5.38 with a value of V_s equal to the base shear obtained from the response spectrum analysis in the direction of interest.

2.5.18.3.2 Nonlinear Time History Analysis

Where a time history analysis procedure is performed, not fewer than three appropriate ground motions shall be used in the analysis as described below.

Ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. If required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground-motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between $0.5T_D$ and $1.25T_M$ (where T_D and T_M are defined in Section 2.5.18.2.1) the average of the SRSS spectra from all horizontal component pairs does not fall below 1.3 times the corresponding ordinate of the design response spectrum (Section 2.5.18.4), by more than 10 percent.

Each pair of ground motion components shall be applied simultaneously to the model considering the most disadvantageous location of eccentric mass. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal displacements at each time step.

The parameters of interest shall be calculated for each ground motion used for the time history analysis. If at least seven ground motions are used for the time history analysis, the average value of the response parameter of interest is permitted to be used for design. If fewer than seven ground motions are analyzed, the maximum value of the response parameter of interest shall be used for design.

2.5.18.3.3 Storey Drift

Maximum story drift corresponding to the design lateral force including displacement due to vertical deformation of the isolation system shall not exceed the following limits:

1. The maximum story drift of the structure above the isolation system calculated by response spectrum analysis shall not exceed $0.015h_{sx}$.
2. The maximum story drift of the structure above the isolation system calculated by nonlinear time history analysis shall not exceed $0.020h_{sx}$.

The storey drift shall be calculated as in Section 2.5.9.7 except that C_d for the isolated structure shall be taken equal to R_I and importance factor equal to 1.0.

2.5.18.4 TESTING

The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures shall be based on test results of isolator units. The tests are for establishing and validating the design properties of the isolation system and shall not be considered as satisfying the manufacturing quality control tests.

The following sequence of tests shall be performed on isolator units for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half the effects due to live load on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.
2. Three fully reversed cycles of loading at each of the following increments of the total design displacement— $0.25D_D$, $0.5D_D$, $1.0D_D$, and $1.0D_M$ where D_D and D_M are as determined in Sections 2.5.18.2.1.
3. Three fully reversed cycles of loading at the total maximum displacement, $1.0D_{TM}$.
4. Not less than ten fully reversed cycles of loading at 1.0 times the total design displacement, $1.0D_{TD}$.

For each cycle of each test, the force-deflection and hysteretic behavior of each isolator unit shall be recorded. The effective stiffness is obtained as the secant value of stiffness at design displacement while the effective damping is determined from the area of hysteretic loop at the design displacement.

2.5.18.5 DESIGN REVIEW

A design review of the isolation system and related test programs shall be performed by an independent team of design professionals experienced in seismic analysis methods and the application of seismic isolation. Isolation system design review shall include, but need not be limited to, the following:

1. Review of site-specific seismic criteria including the development of site-specific spectra and ground motion time histories and all other design criteria developed specifically for the project;
2. Review of the preliminary design including the determination of the total design displacement of the isolation system and the lateral force design level;

3. Overview and observation of prototype (isolator unit) testing
4. Review of the final design of the entire structural system and all supporting analyses; and
5. Review of the isolation system quality control testing program.

2.5.19 BUILDINGS WITH SOFT STOREY

Buildings with possible soft storey action at ground level for providing open parking spaces belong to structures with major vertical irregularity (Fig.2.5.5a). Special arrangement is needed to increase the lateral strength and stiffness of the soft/open storey. The following two approaches may be considered:

1. Dynamic analysis of such building may be carried out incorporating the strength and stiffness of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.
2. Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys. Structural elements (e.g columns and beams) of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads neglecting effect of infill walls. Shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible are to be designed exclusively for 1.5 times the lateral shear force calculated before.

2.5.20 NON-BUILDING STRUCTURES

Calculation of seismic design forces on non-building structures (e.g. chimney, self supported overhead water/fluid tank, silo, trussed tower, storage tank, cooling tower, monument and other structures not covered in the preceding part of Section 2.5) shall be in accordance with "*Chapter 15: Seismic Design Requirements for Non-Building Structures, Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05*" complying with the requirements of Section 2.5 of this Code.

2.6 MISCELLANEOUS LOADS

2.6.1 GENERAL

The procedures and limitations for the determination of selected miscellaneous loads are provided in this section. Loads that are not specified in this section or elsewhere in this chapter, may be determined based on information from reliable references or specialist advice may be sought.

2.6.2 DEFINITIONS

The following definitions and notation shall apply to the provisions of this section only.

ESSENTIAL FACILITIES: Buildings and structures which are necessary to remain functional during an emergency or a post disaster period.

RATIONAL ANALYSIS: An analysis based on established methods or theories using mathematical formulae and actual or appropriately assumed data.

SITE-SPECIFIC DATA: Data obtained either from measurements taken at a site or from substantiated field information required specifically for the structure concerned.

2.6.3 RAIN LOADS

Rain loads shall be determined in accordance with the following provisions.

2.6.3.1 BLOCKED DRAINS

Each portion of a roof shall be designed to sustain the load from all rainwater that could be accumulated on it if the primary drainage system for that portion is undersized or blocked. Ponding instability shall be considered in this situation.

2.6.3.2 CONTROLLED DRAINAGE

Roofs equipped with controlled drainage provisions shall be designed to sustain all rainwater loads on them to the elevation of the secondary drainage system plus 0.25 kN/m^2 . Ponding instability shall be considered in this situation.

2.6.4 LOADS DUE TO FLOOD AND SURGE

For the determination of flood and surge loads on a structural member, consideration shall be given to both hydrostatic and hydrodynamic effects. Required loading shall be determined in accordance with the established principles of mechanics based on site specific criteria and in compliance with the following provisions of this section. For essential facilities like cyclone and flood shelters and for hazardous facilities specified in Table 1.2.1, values of maximum flood elevation, surge height, wind velocities etc., required for the determination of flood and surge load, shall be taken corresponding to 100-year return period. For structures other than essential and hazardous facilities, these values shall be based on 50-year return period.

2.6.4.1 FLOOD LOADS ON STRUCTURES AT INLAND AREAS:

For structures sited at inland areas subject to flood, loads due to flood shall be determined considering hydrostatic effects which shall be calculated based on the flood elevation of 50-year return period. For river-side structures such as that under Exposure C specified in Sec 2.4.8.3, hydrodynamic forces, arising due to approaching wind-generated waves shall also be determined in addition to the hydrostatic load on them. In this case, the amplitude of such wind-induced water waves shall be obtained from site-specific data.

2.6.4.2 FLOOD AND SURGE LOADS ON STRUCTURES AT COASTAL AREAS:

For structures sited at coastal areas, the hydrostatic and hydrodynamic loads shall be determined as follows :

2.6.4.2.1 Hydrostatic Loads

The hydrostatic loads on structural elements and foundations shall be determined based on the maximum static height of water, H_m produced by floods or surges as given by the relation :

$$H_m = \max (h_s, h_f) \quad (2.6.1)$$

$$\text{where, } h_f = y_T - y_g \text{ and} \quad (2.6.2)$$

$$h_s = \text{Maximum surge height as specified in a(i) below.}$$

y_T = Elevation of the extreme surface water level corresponding to a T -year return period specified in (ii) below, metres

y_g = Elevation of ground level at site, metres.

i) Maximum Surge Height, h_s : The maximum surge height, h_s , associated with cyclones, shall be that corresponding to a 50-year or a 100-year return period as may be applicable, based on site specific analysis. In the absence of a more rigorous site specific analysis, the following relation may be used :

$$h_s = h_T - (x - 1) k \quad (2.6.3)$$

where, h_T = design surge height corresponding to a return period of T -years at sea coast, in metres, given in Table 2.6.1.

x = distance of the structure site measured from the spring tide high-water limit on the sea coast, in km; $x = 1$, if $x < 1$.

k = rate of decrease in surge height in m/km; the value of k may be taken as 1/2 for Chittagong-Cox's Bazar-Teknaf coast and as 1/3 for other coastal areas.

ii) Extreme Surface Water Level, y_T : The elevation of the extreme surface water level, y_T for a site, which may not be associated with a cyclonic storm surge, shall be that obtained from a site specific analysis corresponding to a 50-year or a 100-year return period. Values of y_T are given in Table 2.6.2 for selected coastal locations which may be used in the absence of any site specific data.

2.6.4.2.2 Hydrodynamic Loads:

The hydrodynamic load applied on a structural element due to wind-induced local waves of water, shall be determined by a rational analysis using an established method and based on site specific data. In the absence of a site-specific data the amplitude of the local wave, to be used in the rational analysis, shall be taken as $h_w = h_s/4 \geq 1\text{m}$, where, h_s is given in Sec 2.6.4.2.1. Such forces shall be calculated based on 50-year or 100-year return period of flood or surge. The corresponding wind velocities shall be 260 km/h or 289 km/h respectively.

2.6.4.3 BREAKAWAY WALLS

When non-structural walls, partitions or other non-structural elements located below the maximum flood or surge elevation, are required to break away under high tides or wave action, such non-structural elements shall be designed to sustain a maximum uniformly distributed load of 1.0 kN/m^2 but not less than 0.5 kN/m^2 applied on a vertical projection of the area.

2.6.5 TEMPERATURE EFFECTS

Temperature effects, if significant, shall be considered in the design of structures or components thereof in accordance with the provision of this section.

In determining the temperature effects on a structure, the following provisions shall be considered :

a) The temperatures indicated, shall be the air temperature in the shade. The range of the variation in temperature for a building site shall be taken into consideration.

Table 2.6.1 : Design Surge Heights at the Sea Coast, h_T^*

Coastal Region	Surge Height at the Sea Coast, h_T (m)	
	$T = 50\text{-year}^{(1)}$	$T = 100\text{-year}^{(2)}$
Teknaf to Cox's Bazar	4.5	5.8
Chakaria to Anwara, and Maheshkhali-Kutubdia Islands	7.1	8.6
Chittagong to Noakhali	7.9	9.6
Sandwip, Hatiya and all islands in this region	7.9	9.6
Bhola to Barguna	6.2	7.7
Sarankhola to Shyamnagar	5.3	6.4
* Values prepared from information obtained from Annex-D3, MCSP.		
Note : (1) These values may be used in the absence of site specific data for structures other than essential facilities listed in Table 6.1.1.		
(2) These values may be used in the absence of site specific data for essential facilities listed in Table 1.2.1.		

b) Effects of the variation of temperature within the material of a structural element shall be accounted for by one of the following methods.

- i) relieve the stresses by providing adequate numbers of expansion or contraction joints,
- ii) design the structural element to sustain additional stresses due to temperature effects.

c) when the method b(ii) above is considered to be applicable, the structural analysis shall take into account the following :

- i) the variation in temperature within the material of the structural element, exposure condition of the element and the rate at which the material absorb or radiate heat.
- ii) the warping or any other distortion caused due to temperature changes and temperature gradient in the structural element.

d) When it can be demonstrated by established principle of mechanics or by any other means that neglecting some or all of the effects of temperature, does not affect the safety and serviceability of the structure, the temperature effect can be considered insignificant and need not be considered in design.

2.6.6 SOIL AND HYDROSTATIC PRESSURE

For structures or portions thereof, lying below ground level, loads due to soil and hydrostatic pressure shall be determined in accordance with the provisions of this section and applied in addition to all other applicable loads.

Table 2.6.2 : Extreme Surface Water Levels During Monsoon at Selected Locations of the Coastal Area above PWD Datum, y_T^*

Coastal Area		y_T (m)	
Location	Thana	T=50 years ⁽¹⁾	T=100 years ⁽²⁾
Teknaf	Teknaf	2.33	2.44
Cox's Bazar	Cox's Bazar	3.84	3.88
Shaflapur	Moheshkhali	4.67	4.87
Lemsikhali	Kutubdia	4.95	5.19
Banigram	Patiya	5.05	5.24
Chittagong	Bandar	4.72	4.88
Patenga	Bandar	4.08	4.16
Sonapur	Sonagazi	7.02	7.11
Sandwip	Sandwip	6.09	6.2
Companyganj	Companyganj	7.53	7.94
Hatiya	Hatiya	5.55	5.76
Daulatkhan	Daulatkhan	4.62	4.72
Dashmina	Dashmina	3.60	3.73
Galachipa	Galachipa	3.79	3.92
Patuakhali	Patuakhali	2.87	3.03
Khepupara	Kalapara	2.93	3.02
Bamna	Bamna	3.32	3.37
Patharghata	Patharghata	3.65	3.84
Raenda	Sarankhola	3.66	3.75
Chardouni	Patharghata	4.41	4.66
Mongla	Monglaport	3.23	3.36
Kobodak	Shyamnagar	3.51	3.87
(river estuary)			
Kaikhali	Shyamnagar	3.94	4.12
* Values prepared from information obtained from Annex -D3, MCSP			
Note : (1) These values may be used in the absence of site specific data for structures in Structure Occupancy Category IV listed Table 1.2.1.			
(2) These values may be used in the absence of site specific data for structures in Structure Occupancy Categories I, II and III listed in Table 1.2.1.			

2.6.6.1 PRESSURE ON BASEMENT WALL:

In the design of basement walls and similar vertical or nearly vertical structures below grade, provision shall be made for the lateral pressure of adjacent soil. Allowance shall be made for possible surcharge due to fixed or moving loads. When a portion or the whole of the adjacent soil is below the surrounding water table, computations shall be based on the submerged unit weight of soil, plus full hydrostatic pressure.

2.6.6.2 UPLIFT ON FLOORS:

In the design of basement floors and similar horizontal or nearly horizontal construction below grade, the upward pressure of water, if any, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic head shall be measured from the underside of the construction.

2.6.7 LOADS DUE TO EXPLOSIONS

Loads on buildings or portions thereof, shall be assessed in accordance with the provisions of this section.

2.6.7.1 EXPLOSION EFFECTS IN CLOSED ROOMS:

a) Determination of Loads and Response : Internal overpressure developed from an internal explosion such as that due to leaks in gas pipes, evaporation of volatile liquids, internal dust explosion etc., in rooms of sizes comparable to residential rooms and with ventilation areas consisting of window glass breaking at a pressure of 4 kN/m^2 (3-4 mm machine made glass) may be calculated from the following method :

- i) The overpressure, q_o provided in Fig 2.6.1(a) shall be assumed to depend on a factor A_o/v , where, A_o is the total window area in m^2 and v is the volume in m^3 of the room considered,
- ii) The internal pressure shall be assumed to act simultaneously upon all walls and floors in one closed room, and
- iii) The action q_o obtained from Fig 2.6.1(a) may be taken as static action.

When a time dependent response is required, an impulsive force function similar to that shown in Fig 2.6.1(b) shall be used in a dynamic analysis, where t_1 is the time from the start of combustion until maximum pressure is reached and t_2 is the time from maximum pressure to the end of combustion. For t_1

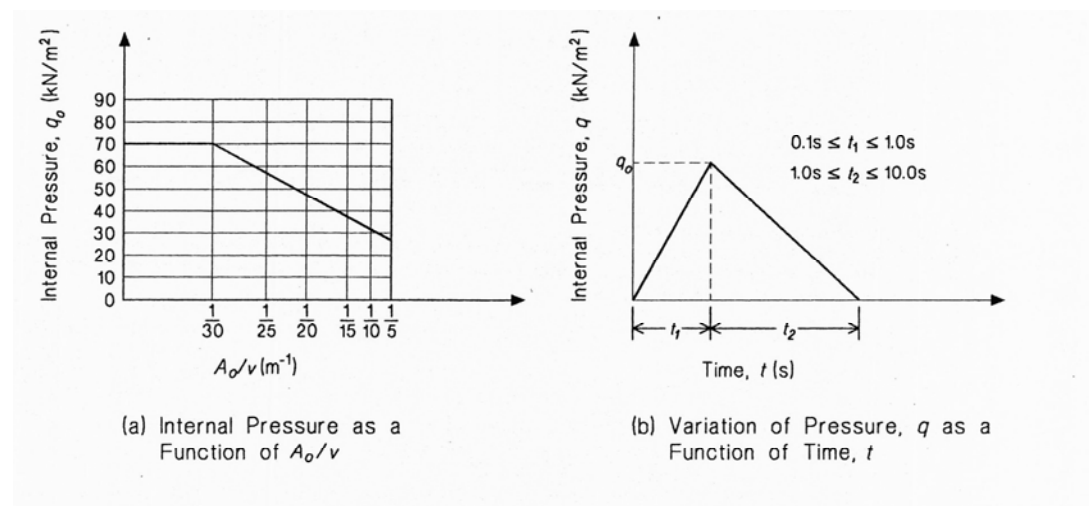


Fig. 2.6.1 Magnitude and distribution of internal pressure in a building due to internal gas explosion

and t_2 the most unfavourable values shall be chosen in relation to the dynamic properties of the structures. However, the values shall be chosen within the intervals as given in Fig 2.6.1(b).

The pressure may be applied solely in one room or in more than one room at the same time. In the latter case, all rooms are incorporated in the volume v . Only windows or other similarly weak and light weight

structural elements may be taken as ventilation areas even though certain limited structural parts break at pressures less than q_o .

b) Limitations : Procedure for determining explosion loads given in (a) above shall have the following limitations:

- i) Values of q_o given in Fig 2.6.1(a) are based on tests with gas explosions in room corresponding to ordinary residential flats, and may be applied to considerably different conditions with caution after appropriate adjustment of the values based on more accurate information.
- ii) Fig 2.6.1 shall be taken as a guide only, and probability of occurrence of an explosion shall be checked in each case using appropriate values.

2.6.7.2 MINIMUM DESIGN PRESSURE:

Walls, floors and roofs and their supporting members separating a use from an explosion exposure, shall be designed to sustain the anticipated maximum load effects resulting from such use including any dynamic effects, but for a minimum internal pressure or suction of 5 kN/m^2 , in addition to all other loads specified in this chapter.

2.6.7.3 DESIGN PRESSURE ON RELIEF VENTS:

When pressure-relief vents are used, such vents shall be designed to relieve at a maximum internal pressure of 1.0 kN/m^2 .

2.6.7.4 LOADS DUE TO OTHER EXPLOSIONS:

Loads arising from other types of explosions, such as those from external gas cloud explosions, external explosions due to high explosives (TNT) etc. shall be determined, for specific cases, by rational analyses based on information from reliable references or specialist advice shall be sought.

2.6.8 VERTICAL FORCES ON AIR RAID SHELTERS

For the design of air raid shelters located in a building e.g. in the basement below ground level, the characteristic vertical load shall be determined in accordance with provisions of Sec 2.6.8.1 below.

Table 2.6.3: Characteristic Vertical Loads for an Air Raid Shelter in a Building

No. of Storeys ⁽¹⁾ Above the Air Raid Shelter	Vertical Load kN/m^2
≤ 2	28
3 - 4	34
≥ 4	41
Buildings of particularly stable construction irrespective of the number of storeys	28 ⁽²⁾
Note : Storeys shall mean every usable storey above the shelter floor	
(1) Buildings of particularly stable construction shall mean buildings	
(2) having bearing structural elements made from reinforced in-situ concrete.	

2.6.8.1 CHARACTERISTIC VERTICAL LOADS

Buildings in which the individual floors are acted upon by a total distributed live load of up to 5.0 kN/m², vertical forces on air raid shelters generally located below ground level, such as a basement, shall be considered to have the characteristic values provided in Table 2.6.3. In the case of buildings having floors that are acted upon by a live load larger than 5.0 kN/m², above values shall be increased by the difference between the average live loads on all storeys above the one used as the shelter and 5.0 kN/m².

2.6.9 LOADS ON HELICOPTER LANDING AREAS

In addition to all other applicable loads provided in this chapter, including the dead load, the minimum live load on helicopter landing or touch down areas shall be one of the loads L_1 , L_2 or L_3 as given below producing the most unfavourable effect :

$$\text{i) } L_1 = W_1 \quad (2.6.4a)$$

$$\text{ii) } L_2 = kW_2 \quad (2.6.4b)$$

$$\text{iii) } L_3 = w \quad (2.6.4c)$$

where, W_1	=	Actual weight of the helicopter in kN,
W_2	=	Fully loaded weight of the helicopter in kN,
w	=	A distributed load of 5.0 kN/m ² ,
k	=	0.75 for helicopters equipped with hydraulic - type shock absorbers, and
	=	1.5 for helicopters with rigid or skid-type landing gear.

The live load, L_1 shall be applied over the actual areas of contact of landing. The load, L_2 shall be a single concentrated load including impact applied over a 300 mm x 300 mm area. The loads L_1 and L_2 may be applied anywhere within the landing area to produce the most unfavourable effects of load.

2.6.10 ERECTION AND CONSTRUCTION LOADS

All loads required to be sustained by a structure or any portion thereof due to placing or storage of construction materials and erection equipment including those due to operation of such equipment shall be considered as erection loads. Provisions shall be made in design to account for all stresses due to such loads.

2.7 COMBINATIONS OF LOADS

2.7.1 GENERAL

Buildings, foundations and structural members shall be investigated for adequate strength to resist the most unfavourable effect resulting from the various combinations of loads provided in this section. The combination of loads may be selected using the provisions of either Sec 2.7.4 or 2.7.5 whichever is applicable. However, once Sec 2.7.4 or 2.7.5 is selected for a particular construction material, it must be used exclusively for proportioning elements of that material throughout the structure. In addition to the load combinations given in Sec 2.7.4 and 2.7.5 any other specific load combination provided elsewhere in this Code shall also be investigated to determine the most unfavourable effect.

The most unfavourable effect of loads may also occur when one or more of the contributing loads are absent, or act in the reverse direction. Loads such as F , H or S shall be considered in design when their effects are significant. Floor live loads shall not be considered where their inclusion results in lower stresses in the member under consideration. The most unfavourable effects from both wind and earthquake loads shall be considered where appropriate, but they need not be assumed to act simultaneously.

2.7.2 DEFINITIONS

ALLOWABLE STRESS DESIGN METHOD (ASD) : A method for proportioning structural members such that the maximum stresses due to service loads obtained from an elastic analysis does not exceed a specified allowable value. This is also called Working Stress Design Method (WSD).

DESIGN STRENGTH : The product of the nominal strength and a resistance factor.

FACTORED LOAD : The product of the nominal load and a load factor.

LIMIT STATE : A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

LOAD EFFECTS : Forces, moments, deformations and other effects produced in structural members and components by the applied loads.

LOAD FACTOR : A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect.

LOADS : Forces or other actions that arise on structural systems from the weight of all permanent constructions, occupants and their possessions, environmental effects, differential settlement, and restrained dimensional changes. Permanent loads are those loads in which variations in time are rare or of small magnitude. All other loads are variable loads.

NOMINAL LOADS : The magnitudes of the loads such as dead, live, wind, earthquake etc. specified in Sec 2.2 through 2.6 of this chapter.

NOMINAL STRENGTH : The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modelling effects and differences between laboratory and field conditions.

RESISTANCE FACTOR : A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure. This is also known as strength reduction factor.

STRENGTH DESIGN METHOD : A method of proportioning structural members using load factors and resistance factors satisfying both the applicable limit state conditions. This is also known as Load Factor Design Method (LFD) or Ultimate Strength Design Method (USD).

WORKING STRESS DESIGN METHOD (WSD) : See ALLOWABLE STRESS DESIGN METHOD.

2.7.3 SYMBOLS AND NOTATION

D = dead loads, or related internal moments and forces, Dead load consists of : a) weight of the member itself, b) weight of all materials of construction incorporated into the building to be permanently supported by the member, including built-in partitions, c) weight of permanent equipment.

E = load effects of earthquake, or related internal moments and forces, For specific definition of the earthquake load effect E , see Section 2.5

F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights or related internal moments and forces.

F_a = loads due to flood or tidal surge or related internal moments and forces.

H = loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces,

L = live loads due to intended use and occupancy, including loads due to movable objects and movable partitions and loads temporarily supported by the structure during maintenance, or related internal moments and forces,. L includes any permissible reduction. If resistance to impact loads is taken into account in design, such effects shall be included with the live loads L .

L_r = roof live loads, or related internal moments and forces,

R = rain load, or related internal moments and forces

T = self-straining forces and cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, or combinations thereof, or related internal moments and forces.

W = wind load, or related internal moments and forces,

2.7.4 COMBINATIONS OF LOAD EFFECTS FOR ALLOWABLE STRESS DESIGN METHOD

2.7.4.1 BASIC COMBINATIONS

Provisions of this section shall apply to all construction materials permitting their use in proportioning structural members by allowable stress design method. When this method is used in designing structural members, all loads listed herein shall be considered to act in the following combinations. The combination that produces the most unfavourable effect shall be used in design.

1. D
2. $D + L$
3. $D + F$
4. $D + H + F + L + T$
5. $D + H + F + (L_r \text{ or } R)$
6. $D + H + F + 0.75(L + T) + 0.75(L_r \text{ or } R)$
7. $D + H + F + (W \text{ or } 0.7E)$

$$8. \quad D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } R)$$

$$9. \quad D + L + (W \text{ or } 0.7E)$$

$$10. \quad 0.6D + W + H$$

$$11. \quad 0.6D + 0.7E + H$$

When a structure is located in a flood zone or in tidal surge zone, the following load combinations shall be considered:

1. In Coastal Zones vulnerable to tidal surges, $1.5F_o$ shall be added to other loads in combinations (7), (8), (9), and (10) and E shall be set equal to zero in (7), (8) and (9).

2. In non-coastal Zones, $0.75F_o$ shall be added to combinations (7), (8), (9), and (10) and E shall be set equal to zero in (7), (8) and (9).

2.7.4.2 STRESS INCREASE

Unless permitted elsewhere in this Code, increases in allowable stress shall not be used with the loads or load combinations given above in Section 2.7.4.1.

2.7.5 COMBINATIONS OF LOAD EFFECTS FOR STRENGTH DESIGN METHOD

When strength design method is used, structural members and foundations shall be designed to have strength not less than that required to resist the most unfavorable effect of the combinations of factored loads listed in the following sections :

2.7.5.1 BASIC COMBINATIONS

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } P)$
3. $1.2D + 1.6(L_r \text{ or } P) + (1.0L \text{ or } 0.8W)$
4. $1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } P)$
5. $1.2D + 1.0E + 1.0L$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Exception :

1. The load factor on live load L in combinations (3), (4), and (5) is permitted to be reduced to 0.5 for all occupancies in which minimum specified uniformly distributed live load is less than or equal to 5.0 kN/m^2 , with the exception of garages or areas occupied as places of public assembly.

2. The load factor on H shall be set equal to zero in combinations (6) and (7) if the structural action due to H counteracts that due to W or E . Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

3. For structures designed in accordance with the provisions of Chapter 6, Part 6 of this Code (reinforced concrete structures), where wind load W has not been reduced by a directionality factor, it shall be permitted to use $1.3W$ in place of $1.6W$ in (4) and (6) above.

When a structure is located in a flood zone or in tidal surge zone, the following load combinations shall be considered:

1. In Coastal Zones vulnerable to tidal surges, $1.6W$ shall be replaced by $1.6W+2.0F_d$ in combinations (4) and (6).
2. In Non-coastal Zones, $1.6W$ shall be replaced by $0.8W+1.0F_d$ in combinations (4) and (6).

2.7.6 LOAD COMBINATIONS FOR EXTRAORDINARY EVENTS

Where required by the applicable code, standard, or the authority having jurisdiction, strength and stability shall be checked to ensure that structures are capable of withstanding the effects of extraordinary (i.e., low-probability) events, such as fires, explosions, and vehicular impact.

Related Appendix

Appendix A Conversion of Expressions from SI to FPS Units

CHAPTER 3

SOILS AND FOUNDATIONS

3.1 INTRODUCTION

The Soils and Foundations chapter of the code is divided into the following three major parts:

- Part A:** General Requirements, Materials and Foundation Types
- Part B:** Service Load Design Method of Foundations
- Part C:** Additional Considerations in Planning, Design and Construction of Building Foundations.

Part A (General Requirements, Materials and Foundation Types) consists of the following sections:

- Scope
- Terminology
- Site Investigations
- Identification, Classification and Description of Soils
- Geotechnical Investigation report
- Materials
- Types of Foundation

Part B (Service Load Design Method of Foundations) has the sections as under:

- Shallow Foundations
- Geotechnical Design of shallow Foundations
- Geotechnical Design of shallow Foundations
- Field Tests for Driven Piles and Drilled Shafts

Part C (Additional Considerations in Planning, Design and Construction of Building Foundations) deals with the following sections:

- Excavation
- Dewatering
- Slope Stability of Adjoining Buildings

- Fills
- Retaining Walls for Foundations
- Waterproofing and Damp-proofing
- Foundation on Slopes
- Foundations on Fill and Problematic Soils
- Foundation Design for Dynamic Forces
- Geo-hazards for Buildings

PART A: GENERAL REQUIREMENTS, MATERIALS AND FOUNDATION TYPES (Sections 3.2 to 3.8)

3.2 SCOPE

The provisions of this chapter shall be applicable to the design and construction of foundations of buildings and structures for the safe support of dead and superimposed loads without exceeding the allowable bearing stresses, permissible settlements and design capability.

3.3 TERMINOLOGY

For the terms used in this chapter, the following definitions shall apply.

ALLOWABLE LOAD: The maximum load that may be safely applied to a foundation unit, considering both the strength and settlement of the soil, under expected loading and soil conditions.

DESIGN LOAD: The expected un-factored load to a foundation unit.

GROSS PRESSURE: The total pressure at the base of a footing due to the weight of the superstructure and the original overburden pressure.

NET PRESSURE: The gross pressure minus the surcharge pressure i.e. the overburden pressure of the soil at the foundation level.

SERVICE LOAD: The expected unfactored load to a foundation unit.

BEARING CAPACITY: The general term used to describe the load carrying capacity of foundation soil or rock in terms of average pressure that enables it to bear and transmit loads from a structure.

BEARING SURFACE: The contact surface between a foundation unit and the soil or rock upon which the foundation rests.

DESIGN BEARING CAPACITY: The maximum net average pressure applied to a soil or rock by a foundation unit that the foundation soil or rock will safely carry without the risk of both shear failure and permissible settlement. It is equal to the least of the two values of net allowable bearing capacity and safe bearing pressure. This may also be called ALLOWABLE BEARING PRESSURE.

GROSS ALLOWABLE BEARING PRESSURE: The maximum gross average pressure of loading that the soil can safely carry with a factor of safety considering risk of shear failure. This may be calculated by dividing gross ultimate bearing capacity with a factor of safety.

GROSS ULTIMATE BEARING CAPACITY: The maximum average gross pressure of loading at the base of a foundation which initiates shear failure of the supporting soil

ALLOWABLE BEARING CAPACITY: The maximum net average pressure of loading that the soil will safely carry with a factor of safety considering risk of shear failure and the settlement of foundation. This is the minimum of safe bearing capacity and safe bearing pressure.

NET ULTIMATE BEARING CAPACITY: The average net increase of pressure at the base of a foundation due to loading which initiates shear failure of the supporting soil. It is equal to the gross ultimate bearing capacity minus the overburden pressure.

PRESUMPTIVE BEARING CAPACITY: The net approximate pressure prescribed as appropriate for the particular type of ground to be used in preliminary designs of foundations

SAFE BEARING CAPACITY: The maximum average pressure of loading that the soil will safely carry without the risk of shear failure. This may be calculated by dividing net ultimate bearing capacity with a factor of safety.

SAFE BEARING PRESSURE: The maximum average pressure of loading that the soil will safely carry without the risk of permissible settlement.

CAISSON: A deep foundation unit, relatively large section, sunk down (not driven) to the ground. This is also called WELL FOUNDATION.

CLAY MINERAL: A small group of minerals, commonly known as clay minerals, essentially composed of hydrous aluminium silicates with magnesium or iron replacing wholly or in part some of the aluminium.

CLAY SOIL: A natural aggregate of microscopic and submicroscopic mineral grains that are product of chemical decomposition and disintegration of rock constituents. It is plastic in moderate to wide range of water contents.

DOWNDRAW: The transfer of load (drag load) to a deep foundation, when soil settles in relation to the foundation. This is also known as **NEGATIVE SKIN FRICTION**.

DRILLED PIER/DRILLED SHAFT: A deep foundation generally of large diameter shaft usually more than 600 mm and constructed by drilling and excavating into the soil.

EFFECTIVE STRESS/ EFFECTIVE PRESSURE: The pressure transmitted through grain to grain at the contact point through a soil mass is termed as effective stress or effective pressure.

END BEARING: The load being transmitted to the toe of a deep foundation and resisted by the bearing capacity of the soil beneath the toe.

EXCAVATION: The space created by the removal of soil or rock for the purpose of construction.

FACTOR OF SAFETY: The ratio of the ultimate capacity to the design (working) capacity of the foundation unit.

FILL: Manmade deposits of natural earth materials (soil, rock) and/or waste materials.

FOOTING: A foundation constructed of masonry, concrete or other material under the base of a wall or one or more columns for the purpose of spreading the load over a larger area at shallower depth of ground surface.

FOUNDATION: Lower part of the structure which is in direct contact with the soil and transmits loads to the ground.

DEEP FOUNDATION: A foundation unit that provides support for a structure transferring loads by end bearing and/or by shaft resistance at considerable depth below the ground. Generally, the depth is at least five times the least dimension of the foundation.

SHALLOW FOUNDATION: A foundation unit that provides support for a structure transferring loads at a small depth below the ground. Generally, the depth is less than two times the least dimension of the foundation.

FOUNDATION ENGINEER: A graduate Engineer with at least five years of experience in civil engineering particularly in foundation design or construction.

GEOTECHNICAL ENGINEER: Engineer with Master's degree in geotechnical engineering having at least three years of experience in geotechnical design or construction.

GROUND WATER LEVEL/ GROUND WATER TABLE: The level of water at which porewater pressure is equal to atmospheric pressure. It is the top surface of a free body of water (piezometric water level) in the ground.

MAT FOUNDATION: See RAFT.

NEGATIVE SKIN FRICTION: See DOWNDRAG.

OVERCONSOLIDATION RATIO (OCR): The ratio of the preconsolidation pressure (maximum past pressure) to the existing effective overburden pressure of the soil.

PILE: A slender deep foundation unit made of materials such as steel, concrete, wood, or combination thereof that transmits the load to the ground by skin friction, end bearing and lateral soil resistance.

BATTER PILE: The pile which is installed at an angle to the vertical in order to carry lateral loads along with the vertical loads. This is also known as RAKER PILE.

BORED PILE/CAST IN-SITU PILE/REPLACEMENT PILE: A pile formed into a preformed hole of ground, usually of reinforced concrete having a diameter smaller than 600 mm.

DRIVEN PILE/DISPLACEMENT PILE: A pile foundation premanufactured and placed in ground by driving, jacking, jetting or screwing.

LATERALLY LOADED PILE: A pile that is installed vertically to carry mainly the lateral loads.

PILE CAP: A pile cap is a special footing needed to transmit the column load to a group or cluster of piles.

PILE HEAD/PILE TOP: The upper small length of a pile.

PILE SHOE: A separate reinforcement or steel form attached to the bottom end (pile toe) of a pile to facilitate driving, to protect the pile toe, and/or to improve the toe resistance of the pile.

PILE TOE/PILE TIP: The bottom end of a pile.

SCREW PILE/ AUGUR PILE: A pre-manufactured pile consisting of steel helical blades and a shaft placed into ground by screwing.

PORE WATER PRESSURE: The pressure induced in the water or vapour and water filling the pores of soil. This is also known as neutral stress.

RAFT: A relatively large spread foundation supporting an arrangement of columns or walls in a regular or irregular layout transmitting the loads to the soil by means of a continuous slab and/or beams, with or without depressions or openings. This is also known as MAT FOUNDATION.

RAKER PILE: See BATTER PILE.

ROCK: A natural aggregate of one or more minerals that are connected by strong and permanent cohesive forces.

ROTATION: It is the angle between the horizontal and any two foundations or two points in a single foundation.

RELATIVE ROTATION/ANGULAR DISTORTION: Angle between the horizontal and any two foundations or two points in a single foundation.

TILT: Rotation of the entire superstructure or at least a well defined part of it.

SETTLEMENT: The downward vertical movement of foundation under load. When settlement occurs over a large area, it is sometimes called subsidence.

CONSOLIDATION SETTLEMENT: A time dependent settlement resulting from gradual reduction of volume of saturated soils because of squeezing out of water from the pores due to increase in effective stress and hence pore water pressure. It is also known as primary consolidation settlement. It is thus a time related process involving compression, stress transfer and water drainage.

DIFFERENTIAL SETTLEMENT: The difference in the total settlements between two foundations or two points in the same foundation.

ELASTIC/DISTORTION SETTLEMENT: It is attributed due to lateral spreading or elastic deformation of dry, moist or saturated soil without a change in the water content and volume.

IMMEDIATE SETTLEMENT: This vertical compression occurs immediately after the application of loading either on account of elastic behaviour that produces distortion at constant volume and on account of compression of air void. For sands, even the consolidation component is immediate.

SECONDARY CONSOLIDATION SETTLEMENT: This is the settlement speculated to be due to the plastic deformation of the soil as a result of some complex colloidal-chemical processes or creep under imposed long term loading.

TOTAL SETTLEMENT: The total downward vertical displacement of a foundation base under load from its as-constructed position. It is the summation of immediate settlement, consolidation settlement and secondary consolidation settlement of the soil.

SHAFT RESISTANCE: The resistance mobilized on the shaft (side) of a deep foundation. Upward resistance is called positive shaft resistance. Downward force on the shaft is called negative shaft resistance.

SOIL: A loose or soft deposit of particles of mineral and/or organic origin that can be separated by such gentle mechanical means as agitation in water.

COLLAPSIBLE SOIL: Consists predominant of sand and silt size particles arranged in a loose honeycomb structure. These soils are dry and strong in their natural state and consolidate or collapse quickly if they become wet.

DISPERSIVE SOIL: Soils that are structurally unstable and disperse in water into basic particles i.e. sand, silt and clay. Dispersible soils tend to be highly erodible. Dispersive soils usually have a high Exchangeable Sodium Percentage (ESP).

EXPANSIVE SOIL: These are clay soils expand when they become wetted and contract when dried. These are formed of clay minerals like montmorillonite and illite.

INORGANIC SOIL: Soil of mineral origin having small amount usually less than 5 percent of organic matter content.

ORGANIC SOIL: Soil having appreciable/significant amount of organic matter content to influence the soil properties.

PEAT SOIL: An organic soil with high organic content, usually more than 75% by weight, composed primarily of vegetable tissue in various stages of decomposition usually with an organic odor, a dark brown to black color, a spongy consistency, and a texture ranging from fibrous to amorphous. Fully decomposed organic soils are known as MUCK.

SOIL PARTICLE SIZE: The sizes of particles that make up soil varying over a wide range. Soil particles are generally gravel, sand, silt and clay, though the terms boulder and cobble can be used to describe larger sizes of gravel.

BOULDER: Particles of rock that will not pass a 12-in. (300-mm) square opening.

Cobbles: Particles of rock that will pass a 12-in. (300-mm) square opening and be retained on a 3-in. (75-mm) sieve.

Clay: A natural aggregate of microscopic and submicroscopic mineral grains less than 0.002 mm in size and plastic in moderate to wide range of water contents.

GRAVEL: Particles of rock that will pass a 3-in. (75-mm) sieve and be retained on a No. 4 (4.75-mm) sieve.

SAND: Aggregates of rounded, sub-rounded, angular, sub-angular or flat fragments of more or less unaltered rock or minerals which is larger than 75 μ m and smaller than 4.75 mm in size.

Silt: Soil passing a No. 200 (75- μ m) sieve that is non-plastic or very slightly plastic and that exhibits little or no strength when air dry.

3.4 SITE INVESTIGATIONS

3.4.1 Sub-Surface Survey

Depending on the type of project thorough investigations has to be carried out for identification, location, alignment and depth of various utilities, e.g., pipelines, cables, sewerage lines, water mains etc. below the surface of the existing ground level. Detailed survey may also be conducted to ascertain the topography of the existing ground.

3.4.2 Sub-Soil Investigations

Subsoil investigation shall be done describing the character, nature, load bearing capacity and settlement capacity of the soil before constructing a new building and structure or for alteration of the foundation of an existing structure.

The aims of a geotechnical investigation are to establish the soil, rock and groundwater conditions, to determine the properties of the soil and rock, and to gather additional relevant knowledge about the site. Careful collection, recording and interpretation of geotechnical information shall be made. This information shall include ground conditions, geology, geomorphology, seismicity and hydrology, as relevant. Indications of the variability of the ground shall be taken into account.

An engineering geological study may be an important consideration to establish the physiographic setting and stratigraphic sequences of soil strata of the area. Geological and agricultural soil maps of the area may give valuable information of site conditions.

During the various phases of sub-soil investigations, e.g. drilling of boreholes, field tests, sampling, groundwater measurements, etc. a competent graduate engineer having experiences in supervising sub-soil exploration works shall be employed by the drilling contractor.

3.4.2.1 Methods of Exploration

Subsoil exploration process may be grouped into three types of activities such as: reconnaissance, exploration and detailed investigations. The reconnaissance method includes geophysical measurements, sounding or probing, while exploratory methods involve various drilling techniques. Field investigations should comprise

- (i) drilling and/or excavations (test pits including exploratory boreholes) for sampling;
- (ii) groundwater measurements;
- (iii) field tests.

Examples of the various types of field investigations are:

- (i) field testing (e.g. CPT, SPT, dynamic probing, WST, pressuremeter tests, dilatometer tests, plate load tests, field vane tests and permeability tests);
- (ii) soil sampling for description of the soil and laboratory tests;
- (iii) groundwater measurements to determine the groundwater table or the pore pressure profile and their fluctuations

- (iv) geophysical investigations (e.g. seismic profiling, ground penetrating radar, resistivity measurements and down hole logging);
- (v) large scale tests, for example to determine the bearing capacity or the behaviour directly on prototype elements, such as anchors.

Where ground contamination or soil gas is expected, information shall be gathered from the relevant sources. This information shall be taken into account when planning the ground investigation. Some of the common methods of exploration, methods of sampling and ground water measurements in soils are described in **Appendix 6.3.A**.

3.4.2.2 Number and Location of Investigation Points

The locations of investigation points, eg., pits and boreholes shall be selected on the basis of the preliminary investigations as a function of the geological conditions, the dimensions of the structure and the engineering problems involved. When selecting the locations of investigation points, the following should be observed:

- (i) the investigation points should be arranged in such a pattern that the stratification can be assessed across the site;
- (ii) the investigation points for a building or structure should be placed at critical points relative to the shape, structural behaviour and expected load distribution (e.g. at the corners of the foundation area);
- (iii) for linear structures, investigation points should be arranged at adequate offsets to the centre line, depending on the overall width of the structure, such as an embankment footprint or a cutting;
- (iv) for structures on or near slopes and steps in the terrain (including excavations), investigation points should also be arranged outside the project area, these being located so that the stability of the slope or cut can be assessed. Where anchorages are installed, due consideration should be given to the likely stresses in their load transfer zone;
- (v) the investigation points should be arranged so that they do not present a hazard to the structure, the construction work, or the surroundings (e.g. as a result of the changes they may cause to the ground and groundwater conditions);
- (vi) the area considered in the design investigations should extend into the neighbouring area to a distance where no harmful influence on the neighbouring area is expected.

Where ground conditions are relatively uniform or the ground is known to have sufficient strength and stiffness properties, wider spacing or fewer investigation points may be applied. In either case, this choice should be justified by local experience.

The locations and spacing of sounding, pits and boreholes shall be such that the soil profiles obtained will permit a reasonably accurate estimate of the extent and character of the intervening soil or rock masses and will disclose important irregularities in subsurface conditions. For building structures, the following guidelines shall be followed:

- (i) For large areas covering industrial and residential colonies, the geological nature of the terrain will help in deciding the number of boreholes or trial pits. The whole area may be divided into grid pattern with Cone Penetration Tests (see **Appendix- 6.3.B**) performed at every 100 m grid points. The number of boreholes or trial pits shall be decided by examining the variation in penetration curves. At least 67% of the required number of borings or trial pits shall be located within the area under the building.
- (ii) In compact building sites covering an area of 0.4 hectare (43,000 square feet), one borehole or trial pit in each corner and one in centre shall be adequate.
- (iii) For widely spaced buildings covering an area of less than 90 m² (1000 square feet) and a height less than four storeys, at least one borehole or trial pit in the centre shall be done.

3.4.2.3 Depth of Exploration

The depth of investigations shall be extended to all strata that will affect the project or are affected by the construction. The depth of exploration shall depend to some extent on the site and type of the proposed structure, and on certain design considerations such as safety against foundation failure, excessive settlement, seepage and earth pressure. Cognizance shall be taken of the character and sequence of the subsurface strata. The site investigation should be carried to such a depth that the entire zone of soil or rock affected by the changes caused by the building or the construction will be adequately explored. A rule of thumb used for this purpose is to extend the borings to a depth where the additional load resulting from the proposed building is less than 10% of the average load of the structure, or less than 5% of the effective stress in the soil at that depth. Where the depth of investigation cannot be related to background information, the following guide lines are suggested to determine the depth of exploration:

- (a) Where substructure units will be supported on spread footings, the minimum depth boring should extend below the anticipated bearing level a minimum of two footing widths for isolated, individual footings where length $\leq$ two times width, and four footing widths for footings where length $>$ five times width. For intermediate footing lengths, the minimum depth of boring may be estimated by linear interpolation as a function of length between depths of two times width and five times width below the bearing level. Greater depth may be required where warranted by local conditions.
- (b) For more heavily loaded structures, such as multistoried structures and for framed structures, at least 50% of the borings should be extended to a depth equal to 1.5 times the width of the building below the lowest part of the foundation.
- (c) Normally the depth of exploration shall be one and a half times the estimated width or the least dimension of the footing below the foundation level. If the pressure bulbs for a number of loaded areas overlap, the whole area may be considered as loaded and exploration shall be carried down to one and a half times the least dimension. In weak soils, the exploration shall be continued to a depth at which the loads can be carried by the stratum in question without undesirable settlement or shear failure.
- (d) Where substructure units will be supported on deep foundations, the depth boring should extend a minimum of 6 m below the anticipated pile or shaft tip elevation. Where pile or shaft groups will be used, the boring should extend at least two times the maximum pile or shaft group dimension below the anticipated tip elevation, unless the foundation will be end bearing on or in rock.
- (e) For piles bearing on rock, a minimum of 1.5 m of rock core should be obtained at each boring location to ensure the boring has not been terminated in a boulder.
- (f) For shafts supported on or extending into rock, a minimum of 1.5 m of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension for a shaft group, whichever is greater, should be obtained to ensure that the boring had not been terminated in a boulder and to determine the physical properties of rock within the zone of foundation influence for design.
- (g) The depth, to which weathering process affects the deposit, shall be regarded as the minimum depth of exploration for a site. However, in no case shall this depth be less than 2 m, but where industrial processes affect the soil characteristics, this depth may be more.
- (h) It is good practice to have at least one boring carried to bedrock, or to well below the anticipated level of influence of the building. Bedrock should be proved by coring into it to a minimum depth of 3 m.

3.4.2.4 Sounding and Penetration Tests

Subsurface soundings are used for exploring soil strata of an erratic nature. They are useful to determine the presence of any soft pockets between drill holes and also to determine the density index of cohesionless soils and the consistency of cohesive soils at desired depths. A field test called Vane Shear Test may be used to determine the shearing strength of the soil located at a depth below the ground.

Penetration tests consist of driving or pushing a standard sampling tube or a cone. The devices are also termed as penetrometers, since they penetrate the subsoil with a view to measuring the resistance to penetrate the soil strata. If a sampling tube is used to penetrate the soil, the test is referred to as Standard Penetration Test (or simply SPT). If a cone is used, the test is called a Cone Penetration Test. If the penetrometer is pushed steadily into the soil, the procedure is known as Static Penetration Test. If driven into the soil, it is known as Dynamic Penetration Test. Details of sounding and penetrations tests are presented in APPENDIX-6.3.A.

3.4.2.5 Geotechnical Investigation Report

The results of a geotechnical investigation shall be compiled in the Geotechnical Investigation Report which shall form a part of the Geotechnical Design Report. The Geotechnical Investigation Report shall consist of the following:

- (i) a presentation of all appropriate geotechnical information on field and laboratory tests including geological features and relevant data;
- (ii) a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.

The Geotechnical Investigation Report shall state known limitations of the results, if appropriate. The Geotechnical Investigation Report should propose necessary further field and laboratory investigations, with comments justifying the need for this further work. Such proposals should be accompanied by a detailed programme for the further investigations to be carried out.

The presentation of geotechnical information shall include a factual account of all field and laboratory investigations. The factual account should include the following information:

- the purpose and scope of the geotechnical investigation including a description of the site and its topography, of the planned structure and the stage of the planning the account is referring to;
- the names of all consultants and contractors;
- the dates between which field and laboratory investigations were performed;
- the field reconnaissance of the site of the project and the surrounding area noting particularly:
 - i) evidence of groundwater;
 - ii) behaviour of neighbouring structures;
 - iii) exposures in quarries and borrow areas;
 - iv) areas of instability;
 - v) difficulties during excavation;
 - vi) history of the site;
 - vii) geology of the site,
 - viii) survey data with plans showing the structure and the location of all investigation points;
 - ix) local experience in the area;
 - x) information about the seismicity of the area.

The presentation of geotechnical information shall include documentation of the methods, procedures and results including all relevant reports of:

- desk studies;
- field investigations, such as sampling, field tests and groundwater measurements;
- laboratory tests.

The results of the field and laboratory investigations shall be presented and reported according to the requirements defined in the ASTM or equivalent standards applied in the investigations.

3.5 IDENTIFICATION, CLASSIFICATION AND DESCRIPTION OF SOILS

3.5.1 Identification of Soil

Samples and trial pits should be inspected visually and compared with field logs of the drillings so that the preliminary ground profile can be established. For soil samples, the visual inspection should be supported by simple manual tests to identify the soil and to give a first impression of its consistency and mechanical behaviour. A standard visual-manual procedure of describing and identifying soils may be followed.

Soil classification tests should be performed to determine the composition and index properties of each stratum. The samples for the classification tests should be selected in such a way that the tests are approximately equally distributed over the complete area and the full depth of the strata relevant for design.

3.5.2 Soil Classification

3.5.2.1 Particle Size Classification

Depending on particle sizes, main soil types are gravel, sand, silt and clay. However, the larger gravels can be further classified as cobble and boulder. The soil particle size shall be classified in accordance with Table 6.3.1.

Table 6.3.1: Particle Size Ranges of Soils

Soil Type		Particle Size Range, mm	Retained on Mesh Size/ Sieve No.
Boulder		> 300	12"
Cobble		300 - 75	3"
Gravel:	Coarse	75 - 19	3/4"
	Medium	19 - 9.5	3/8"
	Fine	9.5 - 4.75	No. 4
Sand:	Coarse	4.75 - 2.00	No. 10
	Medium	2.00 - 0.425	No. 40
	Fine	0.425 - 0.075	No. 200
Silt		0.075 - 0.002	---
Clay		< 0.002	---

3.5.2.2 Engineering Classification

Soils are divided into three major groups, coarse grained, fine grained and highly organic. The classification is based on classification test results namely grain size analysis and consistency test. The coarse grained soils shall be classified using Table 6.3.2. Outlines of organic and inorganic soil separations are also provided in Table 6.3.2. The fine grained

soils shall be classified using the plasticity chart shown in Fig. 6.3.1. For details, reference can be made to ASTM D2487. In addition to these classifications, a soil shall be described by its colour, particle angularity (for coarse grained soils) and consistency. Further to the above classification soils exhibiting swelling or collapsing characteristic shall be recorded.

For undisturbed soils information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics shall be included.

3.5.2.2.1 Identification and Classification of Organic Soils

The presence of organic matter can have undesirable effects on the engineering behaviour of soil. For example, the bearing capacity is reduced, the compressibility is increased, swelling and shrinkage potential is increased due to organic content. Organic content tests are used to classify the soil. In soil with little or no clay particles and carbonate content, the organic content is often determined from the loss on ignition at a controlled temperature. Other suitable tests can also be used. For example, organic content can be determined from the mass loss on treatment with hydrogen peroxide (H_2O_2), which provides a more specific measure of organics. Organic deposits are due to decomposition of organic matters and found usually in topsoil and marshy place. A soil deposit in organic origin is said to peat if it is at the higher end of the organic content scale (75% or more), organic soil at the low end, and muck in between. Peat soil is usually formed of fossilized plant minerals and characterized by fiber content and lower decomposition. The peats have certain characteristics that set them apart from moist mineral soils and required special considerations for construction over them. This special characteristic includes, extremely high natural moisture content, high compressibility including significant secondary and even tertiary compression and very low undrained shear strength at natural moisture content.

However, there are many other criteria existed to classify the organic deposits and it remains still as controversial issue with numerous approaches available for varying purpose of classification. Soil from organic deposits and it refers to a distinct mode of behavior different than traditional soil mechanics in certain respects. A possible approach is being considered by the American society for Testing and Materials for classifying organic soils having varying amount of organic matter contents. The classification is given in Table 6.3.3.

3.5.2.2.2 Identification and Classification of Expansive Soils

Expansive soils are those which swell considerably on absorption of water and shrink on the removal of water. In monsoon seasons, expansive soils imbibe water, become soft and swell. In drier seasons, these soils shrink or reduce in volume due to evaporation of water and become harder. As such, the seasonal moisture variation in such soil deposits around and beneath the structure results into subsequent upward and downward movements of structures leading to structural damage, in the form of wide cracks in the wall and distortion of floors. For identification and classification of expansive soils parameters like free swell, free swell index, linear shrinkage, swelling potential, swelling pressure and volume change should be evaluated experimentally or from available geotechnical correlation.

3.5.2.2.2 Identification and Classification of Collapsible Soils

Soil deposits most likely to collapse are; (i) loose fills, (ii) altered wind-blown sands, (iii) hill wash of loose consistency, and (iv) decomposed granite or other acid igneous rocks.

A very simple test for recognizing collapsible soil is the "sauges test". Two undisturbed cylindrical samples (sausages) of the same diameter and length (volume) are carved from the soil. One sample is then wetted and kneaded to form a cylinder of the original diameter. A decrease in length as compared to the original, undisturbed cylinder will confirm a collapsible grain structure. Collapse is probable when the natural void ratio, e_i is higher than a critical void ratio, e_c that depends on void ratios e_L and e_p at liquid limit and plastic limits respectively.

Table 6.3.2: Engineering Classification of Soils (Criteria for Assigning Group Symbols and Group Names using Laboratory Tests ^A)

Classification (For particles smaller than 75 mm and based on estimated weights)			Group Symbol	Group Name ^B	Laboratory Classification	
					Percent finer than 0.075mm	Other Criteria
Coarse grained soils (More than 50% of the material retained on No. 200 sieve (0.075 mm))	Gravels (More than 50% of coarse fraction retained on No. 4 sieve (4.75 mm))	Clean gravels	GW	Well graded gravels, sandy gravels, sand gravel mixture, little or no fines. ^D	< 5 ^E	$C_u \geq 4$ and $1 \leq C_z \leq 3$ ^C
			GP	Poorly graded gravels, sandy gravels, Sand gravel mixture, little or no fines. ^D		$C_u < 4$ and/or $1 > C_z > 3$ ^C
		Gravel with fines	GM	Silty gravels, silty sandy gravels. ^{D, F, G}	> 12 ^E	Ip < 4 or the limit values below 'A' line of plasticity chart
			GC	Clayey gravels, silty clayey gravels. ^{D, F, G}		For 4 > Ip > 7 and limit values above 'A' line, dual symbol required*
	Sands (over 50% of coarse fraction smaller than 4.75 mm)	Clean Sands	SW	Well graded sand, gravelly sand, little or no fines. ^H	< 5 ^E	$C_u \geq 6$ and $1 \leq C_z \leq 3$ ^C
			SP	Poorly graded sands, gravelly sand, little or no fines. ^H		$C_u < 6$ and/or $1 > C_z > 3$ ^C
		Sands with fines	SM	Silty sand, poorly graded sand silt mixtures. ^{F, G, H}	> 12 ^E	Ip < 4 or the limit values below 'A' line of Plasticity chart
			SC	Clayey sand, sand clay mixtures. ^{F, G, H}		For 4 > Ip > 7 and limit values above A-line, dual symbols required.
Fine grained soils (Over 50% of the material smaller than 0.075 mm)	Silts & Clays $w_L < 50$	Inorganic	ML	Silt of low to medium compressibility, very fine sands, rock flour, silt with sand. ^{K, L, M}	Limit values on or below 'A' line of plasticity chart & Ip < 4	
			CL	Clays of low to medium plasticity, gravelly clay, sandy clay, silty clay, lean clay. ^{K, L, M}	Limit values above 'A' line of plasticity chart and/or Ip > 4	
		Organic	OL	Organic clay ^{K, L, M, N} and Organic silt ^{K, L, M, O} of low to medium plasticity	$\frac{\text{Liquid limit (oven dried)}}{\text{Liquid limit (undried)}} < 0.75$	
	Silts & Clays $w_L \geq 50$	Inorganic	MH	Silt of high plasticity, micaceous fine sandy or silty soil, elastic silt. ^{K, L, M}	Limit values on or below 'A' line of plasticity chart	
			CH	High plastic clay, fat clay. ^{K, L, M}	Limit values above 'A' line of plasticity chart	
		Organic	OH	Organic clay of high plasticity. ^{K, L, M, P}	$\frac{\text{Liquid limit (oven dried)}}{\text{Liquid limit (undried)}} < 0.75$	
	Soils of high organic origin		PT	Peat and highly organic soils. ^{K, L, M, Q}	Identified by colour, odour, fibrous texture and spongy characteristics.	

NOTES:

- A** Based on the material passing the 3-in. (75-mm) sieve
- B** If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
- C** $C_u = D_{60}/D_{10}$, $C_z = (D_{30})^2 / (D_{10} \times D_{60})$
- D** If soil contains $\geq 15\%$ sand, add "with sand" to group name.
- E** Gravels with 5 to 12 % fines require dual symbols:
- GW-GM well-graded gravel with silt
 - GW-GC well-graded gravel with clay
 - GP-GM poorly graded gravel with silt
 - GP-GC poorly graded gravel with clay
- F** If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
- G** If fines are organic, add "with organic fines" to group name.
- H** If soil contains $\geq 15\%$ gravel, add "with gravel" to group name.
- I** Sands with 5 to 12 % fines require dual symbols:
- SW-SM well-graded sand with silt
 - SW-SC well-graded sand with clay
 - SP-SM poorly graded sand with silt
 - SP-SC poorly graded sand with clay.
- J** If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
- K** If soil contains 15 to 29 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.
- L** If soil contains $\geq 30\%$ plus No. 200, predominantly sand, add "sand" to group name.
- M** If soil contains $\geq 30\%$ plus No. 200, predominantly gravel, add "gravelly" to group name.
- N** $PI \geq 4$ and plots on or above "A" line.
- O** $PI < 4$ or plots below "A" line.
- P** PI plots on or above "A" line.
- Q** PI plots below "A" line.

If desired, the percentages of gravel, sand, and fines may be stated in terms indicating a range of percentages, as follows:

Trace -	Particles are present but estimated to be less than 5 %
Few -	5 to 10 %
Little -	15 to 25 %
Some -	30 to 45 %
Mostly -	50 to 100 %

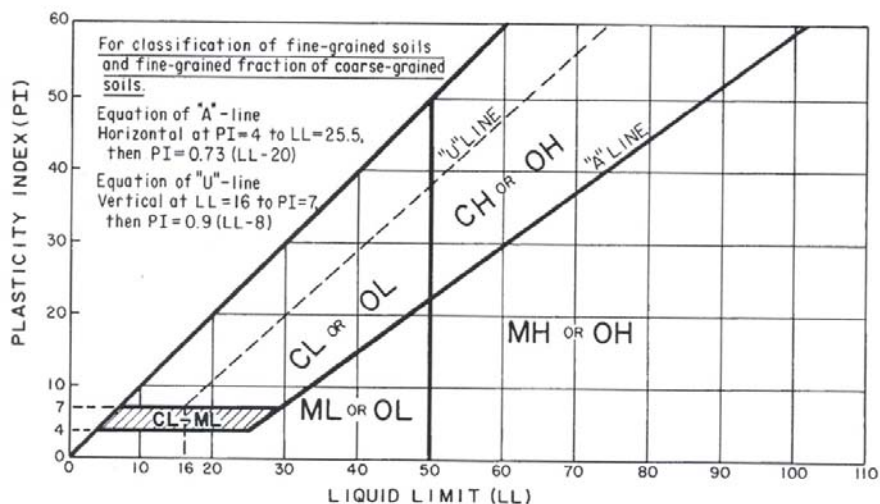


Fig. 6.3.1: Plasticity Chart (based on materials passing 425 µm Sieve)

Table 6.3.3: Classification and Description of Organic Soils (after Edil, 1997)

Organic Content (Test Method : ASTM D2974)	Description
< 5 %	Little effect on behavior; considered inorganic soil.
6 ~ 20 %	Effects properties but behavior is still like mineral soils; organic silts and clays.
21 ~ 74 %	Organic matter governs properties; traditional soil mechanics may be applicable; silty or clayey organic soils.
> 75 %	Displays behavior distinct from traditional soil mechanics especially at low stress.

The following formula should be used to estimate the critical void ratio.

$$e_c = 0.85 e_L + 0.15 e_p \quad (6.3.1)$$

Collapsible soils (with a degree of saturation, $S_r \leq 0.6$) should satisfy the following condition:

$$\frac{e_L - e_i}{1 + e_i} \leq 0.10 \quad (6.3.2)$$

A consolidation test is to be performed on an undisturbed specimen at natural moisture content and to record the thickness, "H" on consolidation under a pressure "p" equal to overburden pressure plus the external pressure likely to be exerted on the soil. The specimen is then submerged under the same pressure and the final thickness H' recorded. Relative subsidence, I_{subs} is found as:

$$I_{subs} = \frac{H - H'}{H} \quad (6.3.3)$$

Soils having $I_{subs} \geq 0.02$ are considered to be collapsible.

3.5.2.2.4 Identification and Classification of Dispersive Soils

Dispersive nature of a soil is a measure of erosion. Dispersive soil is due to the dispersed structure of a soil matrix. An identification of dispersive soils can be made on the basis of pinhole test.

The pinhole test was developed to directly measure dispersibility of compacted fine-grained soils in which water is made to flow through a small hole in a soil specimen, where water flow through the pinhole simulates water flow through a crack or other concentrated leakage channel in the impervious core of a dam or other structure. The test is run under 50, 180, 380 and 1020 mm heads and the soil is classified as follows in Table 6.3.4.

Table 6.3.4: Classification of Dispersive Soil On the Basis of Pinhole Test (Sherard et. al. 1976)

Test Observation	Type of Soil	Class of Soil
Fails rapidly under 50 mm head.	Dispersive soils	D ₁ and D ₂
Erode slowly under 50 mm or 180 mm head	Intermediate soils	ND ₄ and ND ₃
No colloidal erosion under 380 mm or 1020 mm head	Non-dispersive soils	ND ₂ and ND ₁

Another method of identification is to first determine the pH of a 1:2.5 soil/water suspension. If the pH is above 7.8, the soil may contain enough sodium to disperse the mass. Then determine: (i) total exchangable bases, that is, K⁺, Ca₂⁺, Mg₂⁺ and Na⁺ (milliequivalent per 100g of air dried soil) and (ii) cation exchange capacity (CEC) of soil (milliequivalent per 100g of air dried soil). The Exchangeable Sodium Percentage ESP is calculated from the relation:

$$ESP = \frac{N_a}{CEC} \times 100(\%) \quad (6.3.4)$$

Em_gP is given by:

$$EMgP = \frac{Mg}{CEC} \times 100(\%) \quad (6.3.5)$$

If the ESP is above 8 percent and ESP plus EMgP is above 15, dispersion will take place. The soils with ESP=7 to 10 are moderately dispersive in combination with reservoir waters of low dissolved salts. Soils with ESP greater than 15 have serious piping potential. Dispersive soils do not actually present any problems with building structures. However, dispersive soil can lead to catastrophic failures of earth embankment dams as well as severe distress of road embankments.

3.5.2.2.5 Identification and Classification of Soft Inorganic Soils

No standard definition exists for soft clays in terms of conventional soil parameters, mineralogy or geological origin. It is, however, commonly understood that soft clays give shear strength, compressibility and severe time related settlement problems. In near surface clays, where form a crust, partial saturation and overconsolidation occur together and the overconsolidation is a result of the drying out of the clay due to changes in the water table.

In below surface clays, overconsolidation may have taken place when the clay was previously at, or close to the ground surface and above the water table, but due to subsequent deposition the strata may now be below the surface, saturated and overconsolidated. Partial saturation does not in itself cause engineering problems, but may lead to laboratory testing difficulties. Soft clays have undrained shear strengths between about 10kPa and 40kPa, in other words, from exuding between the fingers when squeezed to being easily moulded in the fingers.

Soft clays present very special problems of engineering design and construction. Foundation failures in soft clays are comparatively common. The construction of buildings in soft clays has always been associated with stability problems and settlement. Shallow foundations inevitably results in large settlements which must be accommodated for in the design, and which invariably necessitate long-term maintenance of engineered facilities. The following relationship among N-values obtained from SPT, consistency and undrained shear strength of soft clays may be used as guides.

N-value (blows/300 mm of penetration)	Consistency	Undrained Shear Strength (kN/m ²)
Below 2	Very soft	Less than 20
2 – 4	Soft	20 – 40

Undrained shear strength is half of unconfined compressive strength as determined from unconfined compression test or half of the peak deviator stress as obtained from unconsolidated undrained (UU) triaxial compression test.

3.6 MATERIALS

All materials for the construction of foundations shall conform to the requirements of Part 5: Building Materials.

3.6.1 Concrete

All concrete materials and steel reinforcement used in foundations shall conform to the requirements specified in Chapter 5 unless otherwise specified in this section. For different types of foundation the recommended concrete properties are shown in Table 6.3.5. However, special considerations should be given for hostile environment (salinity, acidic environment).

Table 6.3.5: Properties of Concrete for Different Types of Foundations

Foundation Type	Minimum cement content (kg/m ³)	Specified Min. 28days Cylinder Strength (MPa)	Slump (mm)	Remarks
Footing/raft	350	20	25 to 125	Retarder and plasticizer recommended.
Drilled shaft/ Cast-in-situ pile (tremie concrete)	400	18	125 to 200	
Driven pile	350	25	25 to 125	

3.6.2 Steel

3.7.2.1 General

Corrosion in soil, water or moist out-door environment is caused by electro-chemical processes. The process takes place in corrosion cells on the steel surface, which consists of an anodic surface (where the corrosion takes place), a cathodic surface (where oxygen is reduced) and the electrolyte, which reacts with these surfaces. In the case of general corrosion, the surface erosion is relatively even across the entire surface. Local corrosion however is concentrated to a limited surface area. Pronounced cavity erosion is rather unusual on unprotected carbon steel in soil or water.

In many circumstances, steel corrosion rates are low and steel piles may be used for permanent works in an unprotected condition. The degree of corrosion and whether protection is required depend upon the working environment which can be variable, even within a single installation. Underground corrosion of steel piles driven into undisturbed soils is negligible irrespective of the soil type and characteristics. The insignificant corrosion attack is attributed to the low oxygen levels present in undisturbed soil. For the purpose of calculations, a maximum corrosion rate of 0.015 mm per side per year may be used. In recent-fill soils or industrial waste soils, where corrosion rates may be higher, protection systems should be considered.

3.7.2.2 Atmospheric Corrosion

Atmospheric corrosion of steel in the UK averages approximately 0.035 mm/side per year and this value may be used for most atmospheric environments.

3.7.2.3 Corrosion in Fresh Waters

Corrosion losses in fresh water immersion zones are generally lower than for sea water so the effective life of steel piles is normally proportionately longer. However, fresh waters are variable and no general advice can be given to quantify the increase in the length of life.

3.7.2.4 Corrosion in Marine Environments

Marine environments may include several exposure zones with different aggressivity and different corrosion performance.

- Below the bed level: Where piles are below the bed level little corrosion occurs and the corrosion rate given for underground corrosion is applicable, that is, 0.015 mm/side per year.
- Seawater immersion zone: Corrosion of steel piling in immersion conditions is normally low, with a mean corrosion rate of 0.035 mm/side per year.
- Tidal zones: Marine growths in this zone give significant protection to the piling, by sheltering the steel from wave action between tides and by limiting the oxygen supply to the steel surface. The corrosion rate of steels

in the tidal zone is similar to that of immersion zone corrosion, i.e. 0.035 mm/side per year. Protection should be provided where necessary, to the steel surfaces to prevent the removal or damage of the marine growth.

- (d) Low water zone: In tidal waters, the low water level and the splash zone are reasons of highest thickness losses, where a mean corrosion rate of 0.075 mm/side per year occurs. Occasionally higher corrosion rates are encountered at the lower water level because of specific local conditions.
- (e) Splash and atmospheric zones: In the splash zone, which is a more aggressive environment than the atmospheric zone, corrosion rates are similar to the low water level, i.e. 0.075 mm/side per year. In this zone thick stratified rust layers may develop and at thicknesses greater than 10 mm these tend to spall from the steel especially on curved parts of the piles such as the shoulders and the clutches. Rust has a much greater volume than the steel from which it is derived so that the steel corrosion losses are represented by some 10 % to 20 % of the rust thickness.

The boundary between the splash and atmospheric zones is not well defined, however, corrosion rates diminish rapidly with distance above peak wave height and the mean atmospheric corrosion rate of 0.035 mm/side per year can be used for this zone.

3.7.2.5 Methods of Increasing Effective Life

The effective life of unpainted or otherwise unprotected steel piling depends upon the combined effects of imposed stresses and corrosion. Where measures for increasing the effective life of a structure are necessary, the following should be considered; introduction of a corrosion allowance (i.e. oversized cross-sections of piles, high yield steel etc), anti-corrosion painting, application of a polyethylene (PE) coating (on steel tube piles), zinc coating, electro-chemical (cathodic) protection, casting in cement mortar or concrete, and use of atmospheric corrosion resistant steel products instead of ordinary carbon steel in any foundation work involving steel.

- (a) Use of a heavier section: Effective life may be increased by the use of additional steel thickness as a corrosion allowance. Maximum corrosion seldom occurs at the same position as the maximum bending moment. Accordingly, the use of a corrosion allowance is a cost effective method of increasing effective life. It is preferable to use atmospheric corrosion resistant high strength low alloy steel.
- (b) Use of a high yield steel: An alternative to using mild steel in a heavier section is to use a higher yield steel and retain the same section.
- (c) Zinc coatings: Steel piles should normally be coated under shop conditions. Paints should be applied to the cleaned surface by airless spraying and then cured rapidly to produce the required coating thickness in as few coats as possible. Hot zinc-coating of steel piles in soil can achieve normally long-lasting protection, provided that the zinc layer has sufficient thickness. In some soils, especially those with low pH-values, the corrosion of zinc can be high, thereby shortening the protection duration. Low pH-values occur normally in the aerated zone above the lowest ground water level. In such a case, it is recommended to apply protection paint on top of the zinc layer.
- (d) Concrete encasement: Concrete encasement may be used to protect steel piles in marine environment. The use of concrete may be restricted to the splash zone by extending the concrete cope to below the mean high water level, both splash and tidal zones may be protected by extending the cope to below the lowest water level. The concrete itself should be a quantity sufficient to resist seawater attack.
- (e) Cathodic protection: The design and application of cathodic protection systems to marine piles structures is a complex operation requiring the experience of specialist firms. Cathodic protection with electric current applied to steel sheet pile wall. Rod-type anodes are connected directly with steel sheet pile. Cathodic protection is considered to be fully effective only up to the half-tide mark. For zones above this level,

including the splash zone, alternative methods of protection may be required, in addition to cathodic protection. Where cathodic protection is used on marine structures, provision should be made for earthing ships and buried services to the quay.

- (f) Polyethelene coating: Steel tube piles can be protected effectively by application of a PE-cover of a few millimeter thickness. This cover can be applied in the factory and is usually placed on a coating of epoxy. Steel tube piles in water, where the mechanical wear is low, can in this way be protected for long time periods. When the steel tube piles with the PE-cover are driven into coarse-grained soil, the effect of damaging the protection layer must be taken into consideration.
- (g) Properly executed anti-corrosion measures, using high-quality methods can protect steel piles in soil or water over periods of 15 to 20 years. PE-cover in combination with epoxy coating can achieve even longer protection times.

3.6.3 Timber

Timber may be used only for foundation of temporary structure and shall conform to the standards specified in Sec 2.9 of Part 5. Where timber is exposed to soil or used as load bearing pile above ground water level, it shall be treated in accordance with BDS 819:1975.

3.7 TYPES OF FOUNDATION

3.7.1 Shallow Foundation

Shallow foundations spread the load to the ground at shallow depth. Generally, the capacity of this foundation is derived from bearing.

3.7.1.1 Footing

Footings are foundations that spread the load to the ground at shallow depths. These include individual column footings, continuous wall footings, and combined footings. Footings shall be provided under walls, pilasters, columns, piers, chimneys etc. bearing on soil or rock, except that footings may be omitted under pier or monolithic concrete walls if safe bearing capacity of the soil or rock is not exceeded.

3.7.1.2 Raft/ Mat

A foundation consisting of continuous slab that covers the entire area beneath the structure and supports all walls and columns is considered as a raft or mat foundation. A raft foundation may be one of the following types:

- (a) Flat plate or concrete slab of uniform thickness usually supporting columns spaced uniformly and resting on soils of low compressibility.
- (b) Flat plates as in (a) but thickened under columns to provide adequate shear and moment resistance.
- (c) Two way slab and beam system supporting largely spaced columns on compressible soil.
- (d) Cellular raft or rigid frames consisting of slabs and basement walls, usually used for heavy structures.

3.7.2 Deep Foundation

A cylindrical/box foundation having a ratio of depth to base width greater than 5 is considered a Deep Foundation. Generally, its capacity is derived from friction and end bearing.

3.7.2.1 Driven piles

A slender deep foundation unit made of materials such as steel, concrete, wood, or combination thereof, which is pre-manufactured and placed by driving, jacking, jetting or screwing and displacing the soil.

- (a) Driven Precast Concrete Piles: Pile structure capable of being driven into the ground and able to resist handling stresses shall be used for this category of piles.
- (b) Driven Cast-in-situ Concrete Piles : A pile formed by driving a steel casing or concrete shell in one or more pieces, which may remain in place after driving or withdrawn, with the inside filled with concrete, falls in this category of piles. Sometimes an enlarged base may be formed by driving out a concrete plug.
- (c) Driven Prestressed Concrete Pile: A pile constructed in prestressed concrete in a casting yard and subsequently driven in the ground when it has attained sufficient strength.
- (d) Timber Piles: structural timber (see Sec 2.9 of Part 5) shall be used as piles for temporary structures for directly transmitting the imposed load to soil. When driven timber poles are used to compact and improve the deposit.

3.8.2.2 Bored piles/ cast-in-situ piles

A deep foundation of generally small diameter, usually less than 600 mm, constructed using percussion or rotary drilling into the soil. These are constructed by concreting bore holes formed by auguring, rotary drilling or percussion drilling with or without using bentonite mud circulation. Excavation or drilling shall be carried out in a manner that will not impair the carrying capacity of the foundations already in place or will not damage adjacent foundations. These foundations may be tested for capacity by load test or for integrity by sonic response or other suitable method. Under-reaming drilled piers can be constructed in cohesive soils to increase the end bearing.

3.8.2.3 Drilled pier/ drilled shafts

The drilled pier is a type of bored pile having a larger diameter (more than 600 mm) constructed by excavating the soil or sinking the foundation.

3.8.2.4 Caisson/ well

A caisson or well foundation is a deep foundation of large diameter relative to its length that is generally a hollow shaft or box which is sunk to position. It differs from other types of deep foundation in the sense that it undergoes rigid body movement under lateral load, whereas the others are flexible like a beam under such loads. This type of foundation is usually used for bridges and massive structures.

PART B: SERVICE LOAD DESIGN METHOD OF FOUNDATIONS (SECTIONS 3.9 to 3.12)

3.8 SHALLOW FOUNDATION

Shall be applicable to isolated Footings, Combined Footings and Raft/ Mats.

3.8.1 Distribution of Bearing Pressure

Footing shall be designed to keep the maximum imposed load within the safe bearing values of soil and rock. To prevent unequal settlement footing shall be designed to keep the bearing pressure as nearly uniform as practical.

For raft design, distribution of soil pressures should be consistent with the properties of the foundation materials (subsoil) and the structure (raft thickness) and with the principles of geotechnical engineering. Mat or raft and floating foundations shall only be used when the applied load of building or structure is so arranged as to result in practically uniformly balanced loading, and the soil immediately below the mat is of uniform bearing capacity.

3.8.2 Footings in Fill Soil

Footings located in fill are subject to the same bearing capacity, settlement, and dynamic ground stability considerations as footings in natural soil. The behavior of both fill and underlying natural soil should be considered.

3.8.3 Soil and Rock Property Selection

Soil and rock properties defining the strength and compressibility characteristics of foundation materials are required for footing design. Foundation stability and settlement analysis for design shall be conducted using soil and rock properties based on the results of field and laboratory testing.

3.8.4 Minimum Depth of Foundation

The minimum depth of foundation shall be 1.5 m for exterior footing of permanent structures in cohesive soils and 2 m in cohesionless soils. For temporary structures the minimum depth of exterior footing shall be 400 mm. In case of expansive and soils susceptible to weathering effects, the above mentioned minimum depths will be not applicable and may have to be increased.

3.8.5 Scour

Footings supported on soil shall be embedded sufficiently below the maximum computed scour depth or protected with a scour countermeasure.

3.8.6 Mass Movement of Ground in Unstable Areas

In certain areas mass movement of ground may occur from causes independent of the loads applied to the foundation. These include mining subsidence, landslides on unstable slopes and creep on clay slopes. In areas of ground subsidence, foundations and structures should be made sufficiently rigid and strong to withstand the probable worst loading conditions. The construction of structures on slopes which are suspected of being unstable and subject to landslip shall be avoided. Spread foundations on such slopes shall be on a horizontal bearing and stepped. For foundations on clay slopes, the stability of the foundation should be investigated.

3.8.7 Foundation Excavation

Foundation excavation below ground water table particularly in sand shall be made such that the hydraulic gradient at the bottom of the excavation is not increased to a magnitude that would cause the foundation soils to loosen due to upward flow of water. Further, footing excavations shall be made such that hydraulic gradients and material removal do not adversely affect adjacent structures. Seepage forces and gradients may be evaluated by standard flow net procedures. Dewatering or cutoff methods to control seepage shall be used when necessary.

In case of soil excavation for raft foundations, the following issues should be additionally taken into consideration:

- (a) Protection for the excavation using shore or sheet piles and/or retaining system with or without bracing, anchors etc.
- (b) Consideration of the additional bearing capacity of the raft for the depth of the soil excavated.
- (c) Consideration of the reduction of bearing capacity for any upward buoyancy pressure of water.

3.9 GEOTECHNICAL DESIGN OF SHALLOW FOUNDATIONS

3.9.1 General

Shallow foundations on soil shall be designed to support the design loads with adequate bearing and structural capacity and with tolerable settlements. In addition, the capacity of footings subjected to seismic and dynamic loads shall be appropriately evaluated. The location of the resultant pressure on the base of the footings should be maintained preferably within B/6 of the centre of the footing.

3.9.2 Design Load

Shallow foundation design (considering bearing capacity due to shear strength) shall consider the most unfavourable effect of the following combinations of loading:

- (a) Full Dead Load + Normal Live Load
- (b) Full Dead Load + Normal Live Load + Wind Load or Seismic Load
- (c) $0.9 \times (\text{Full Dead Load}) + \text{Buoyancy Pressure}$

Shallow foundation design (considering settlement) shall consider the most unfavourable effect of the following combinations of loading:

SAND

- (a) Full Dead Load + Normal Live Load
- (b) Full Dead Load + Normal Live Load + Wind Load or Seismic Load

CLAY

Full Dead Load + $0.5 \times$ Normal Live Load

3.9.3 Bearing capacity

When physical characteristics such as cohesion, angle of internal friction, density etc. are available, the bearing capacity shall be calculated from stability considerations. Established bearing capacity equations shall be used for calculating bearing capacity. A factor of safety of between 2.0 to 3.0 (depending on the extent of soil exploration, quality control and monitoring of construction) shall be adopted to obtain allowable bearing pressure when dead load and normal live load is used. Thirty three percent overstressing above allowable pressure shall be allowed in case of design considering wind or seismic loading. Allowable load shall also limit settlement between supporting elements to a tolerable limit.

3.9.4 Presumptive Bearing Capacity for Preliminary Design

For lightly loaded and small sized structures (two storied or less in occupancy category A, B, C & D) and for preliminary design of any structure, the presumptive bearing values (allowable) as given in Table 6.3.6 may be assumed for uniform soil in the absence of test results.

Table 6.3.6: Presumptive Values of Bearing Capacity for Lightly Loaded Structures *

Soil Type	Soil Description	Safe Bearing Capacity, kPa
1	Soft Rock or Shale	440
2	Gravel, sandy gravel, silty sandy gravel; very dense and offer high resistance to penetration during excavation (soil shall include the groups GW, GP, GM, GC)	400 **

Soil Type	Soil Description	Safe Bearing Capacity, kPa
3	Sand (other than fine sand), gravelly sand, silty sand; dry (soil shall include the groups SW, SP, SM, SC)	200**
4	Fine sand; loose & dry (soil shall include the groups SW, SP)	100**
5	Silt, clayey silt, clayey sand; dry lumps which can be easily crushed by finger (soil shall include the groups ML, SC, & MH)	150
6	Clay, sandy clay; can be indented with strong thumb pressure (soil shall include the groups CL, & CH)	150
7	Soft clay; can be indented with modest thumb pressure (soil shall include the groups CL, & CH)	100
8	Very soft clay; can be penetrated several centimeters with thumb pressure (soil shall include the groups CL & CH)	50
9	Organic clay & Peat (soil shall include the groups OH, OL, Pt)	To be determined after investigation.
10	Fills	To be determined after investigation.
* Two stories or less (Occupancy category A, B, C and D as per BNBC) ** 50% of these values shall be used where water table is above the base, or below it within a distance equal to the least dimension of foundation		

3.9.5 Allowable Increase of Bearing Pressure due to Wind and Earthquake Forces

The allowable bearing pressure of the soil determined in accordance with this section may be increased by 33 per cent when lateral forces due to wind or earthquake act simultaneously with gravity loads. No increase in allowable bearing pressure shall be permitted for gravity loads acting alone. In a zone where seismic forces exist, possibility of liquefaction in loose sand, silt and sandy soils shall be investigated.

3.9.6 Settlement of Foundation

Foundation shall be so designed that the allowable bearing capacity is not exceeded, and the total and differential settlement are within permissible values. Foundations can settle in various ways and each affects the performance of the structure. The simplest mode consists of the entire structure settling uniformly. This mode does not distort the structure. Any damage done is related to the interface between the structure and adjacent ground or adjacent structures. Shearing of utility lines could be a problem. Another possibility is that one side of the structure settles much more than the opposite side and the portions in between settle proportionately. This causes the structure to tilt, but it still does not distort. A nominal tilt will not affect the performance of the structure, although it may create aesthetic and public confidence problems. However, as a result of difference in foundation settlement the structure may settle and distort causing cracks in walls and floors, jamming of doors and windows and overloading of structural members.

3.9.7 Total Settlement

Total settlement (δ) is the absolute vertical movement of the foundation from its as-constructed position to its loaded position. Total settlement of foundation due to net imposed load shall be estimated in accordance with established engineering principle. An estimate of settlement with respect to the following shall be made where applicable:

- (i) Elastic compression of the underlying soil below the foundation and of the foundation.
- (ii) Consolidation settlement.
- (iii) Secondary consolidation/compression of the underlying soil.

- (iv) Compression and volume change due to change in effective stress or soil migration associated with lowering or movement of ground water.
- (v) Seasonal swelling and shrinkage of expansive clays.
- (vi) Ground movement on earth slopes, such as surface erosion, creep or landslide.
- (vii) Settlement due to adjacent excavation, mining subsidence and underground erosion.

In normal circumstances of inorganic and organic soil deposits the total settlement is attributed due to the first three factors as mentioned above. The other factors are regarded as special cases. Because soil settlement can have both time-dependent and noontime-dependent components, it is often categorized in terms short-term settlement (or immediate settlement) which occurs as quickly as the load is applied, and long-term settlement (or delayed settlement), which occurs over some longer period. Many engineers associate consolidation settlement solely with the long term settlement of clay. However, this is not strictly true. Consolidation is related to volume change due to change in effective stress regardless of the type of soil or the time required for the volume change.

3.9.7.1 Elastic/ Distortion Settlement

Elastic Settlement (δ_d) of foundation soils results from lateral movements of the soil without volume change in response to changes in effective vertical stress. This is non-time dependent phenomenon and similar to the Poisson's effect where an object is loaded in the vertical direction expands laterally. Elastic or distortion settlements primarily occur when the load is confined to a small area, such as a structural foundation, or near the edges of large loaded area such as embankments.

3.9.7.2 Immediate Settlement/ Short Term Settlement

This vertical compression occurs immediately after the application of loading either on account of elastic behaviour that produces distortion at constant volume and on account of compression of air void. This is sometimes designated as δ_i for sandy soils, even the consolidation component is immediate.

3.9.7.3 Primary Consolidation Settlement

Primary consolidation settlement or simply the consolidation settlement (δ_c) of foundation is due to consolidation of the underlying saturated or nearly saturated soil especially cohesive silt or clay. The full dead load and 50% of the total live load should be considered when computing the consolidation settlement of foundations on clay soils.

3.9.7.4 Secondary Consolidation Settlement

Secondary consolidation settlement (δ_s) of the foundation is due to secondary compression or consolidation of the underlying saturated or nearly saturated cohesive silt or clay. This is primarily due to particle reorientation, creep, and decomposition of organic materials. Secondary compression is always time-dependent and can be significant in highly plastic clays, organic soils, and sanitary landfills, but it is negligible in sands and gravels.

3.9.7.5 Differential Settlement and its Effect on the Structure

Differential settlement is the difference in total settlement between two foundations or two points in the same foundation. It occurs as a result of relative movement between two parts of a building. The related terms describing the effects of differential settlement on the structural as a whole or on parts of it are tilt, rotation and angular distortion/relative rotation which are defined below.

3.9.7.6 Tilt

It is rotation of the entire superstructure or a well defined part of it as a result of non-uniform or differential settlement of foundation as a result of which one side of the building settles more than the other thus affecting the verticality of the building.

3.9.7.7 Rotation

It is the angle between the horizontal line and an imaginary straight line connecting any two foundations or two points in a single foundation.

3.9.7.8 Angular Distortion/Relative Rotation

Angular distortion or relative rotation is the angle between imaginary straight line indicating the overall tilt of a structure and the imaginary connecting line indicating the inclination of a specific part of it. It is measured as the ratio of differential settlement to the distance between the two points. .

3.9.8 Causes of Differential Settlement

Due consideration shall be given to estimate the differential settlement that may occur under the building structure under the following circumstances:

- (i) Nonuniformity in subsoil formation within the area covered by the building due to geologic or man-made causes, or anomalies in type, structure, thickness and density of the formation.
- (ii) Nonuniform pressure distribution due to nonuniform and incomplete loading.
- (iii) Ground water condition during and after construction.
- (iv) Loading influence of adjacent structures.
- (v) Uneven expansion and contraction due to moisture migration, uneven drying, wetting or softening.

3.9.9 Tolerable Settlement , Tilt and Rotation

Allowable or limiting settlement of a building structure will depend on the nature of the structure, the foundation and the soil. Different types of structures have varying degrees of tolerance to settlements and distortions. These variations depend on the type of construction, use of the structure, rigidity of the structure and the presence of sensitive finishes. As a general rule, a total settlement of 25 mm and a differential settlement of 20 mm between columns in most buildings shall be considered safe for buildings on isolated pad footings on sand for working load (unfactored). A total settlement of 40 mm and a differential settlement of 20 mm between columns shall be considered safe for buildings on isolated pad footings on clay soil for working load. Buildings on raft can usually tolerate greater total settlements. Limiting tolerance for distortion and deflections introduced in a structure is necessarily a subjective process, depending on the status of the building and any specific requirements for serviceability. The limiting values, given in Table 6.3.7 may be followed as guidelines.

3.9.10 Dynamic Ground Stability or Liquefaction Analysis

Soil liquefaction is a phenomenon in which a saturated soil deposit loses most, if not all, of its strength and stiffness due to the generation of excess pore water pressure during earthquake-induced ground shaking. It has been a major cause for damage of structures during past earthquakes (e.g., 1964 Niigata Earthquake). Current knowledge of liquefaction is significantly advanced and several evaluation methods are available. Hazards due to liquefaction are routinely evaluated and mitigated in seismically active developed parts of the world.

Table 6.3.7: Permissible Total Settlement, Differential Settlement and Angular Distortion (tilt) for Shallow Foundations in Soils (Adapted from NBCI, 2005)

Type of Structure	Isolated Foundations						Raft Foundation					
	Sand and Hard Clay			Plastic Clay			Sand and Hard Clay			Plastic Clay		
	Maximum Settlement (mm)	Differential Settlement (mm)	Angular Distortion (mm)	Maximum Settlement (mm)	Differential Settlement (mm)	Angular Distortion (mm)	Maximum Settlement (mm)	Differential Settlement (mm)	Angular Distortion (mm)	Maximum Settlement (mm)	Differential Settlement (mm)	Angular Distortion (mm)
Steel Structure	50	0.0033 L	1/300	50	0.0033 L	1/300	75	0.0033 L	1/300	100	0.0033 L	1/300
RCC Structures	50	0.0015 L	1/666	75	0.0015 L	1/666	75	0.0021 L	1/500	100	0.002 L	1/500
Multistoried Building												
(a) RCC or steel framed building with panel walls	60	0.002 L	1/500	75	0.002 L	1/500	75	0.0025 L	1/400	125	0.0033 L	1/300
(b) Load bearing walls												
(i) $L/H = 2$ *	60	0.0002 L	1/5000	60	0.0002 L	1/5000	Not likely to be encountered					
(ii) $L/H = 7$ *	60	0.0004 L	1/2500	60	0.0004 L	1/2500	Not likely to be encountered					
Silos	50	0.0015 L	1/666	75	0.0015 L	1/666	100	0.0025 L	1/400	125	0.0025 L	1/400
Water Tank	50	0.0015 L	1/666	75	0.0015 L	1/666	100	0.0025 L	1/400	125	0.0025 L	1/400
Note: The values given in the Table may be taken only as a guide and the permissible total settlement, differential settlement and tilt (angular distortion) in each case should be decided as per requirements of the designer. L denotes the length of deflected part of wall/ raft or centre to centre distance between columns. H denotes the height of wall from foundation footing. * For intermediate ratios of L/H, the values can be interpolated.												

Liquefaction Analysis

Liquefaction can be analyzed by a simple comparison of the seismically induced shear stress with the similarly expressed shear stress required to cause initial liquefaction or whatever level of shear strain amplitude is deemed intolerable in design. Usually, the occurrence of 5% double amplitude (DA) axial strain is adopted to define the cyclic strength consistent with 100% pore water pressure build-up. The corresponding strength (CRR) can be obtained by several procedures.

Thus, the liquefaction potential of a sand deposit is evaluated in terms of factor of safety F_L , defined as in Equation (6.3.6). The externally applied cyclic stress ratio (CSR) can be evaluated by Equations (6.3.7a, 6.3.7b and 6.3.8).

$$F_L = \frac{CRR}{CSR} \quad (6.3.6)$$

If the factor of safety F_L is ≤ 1 , liquefaction is said to take place. Otherwise, liquefaction does not occur. The factor of safety obtained in this way is generally used to identify the depth to which liquefaction is expected to occur in a future earthquake. This information is necessary if some countermeasure is to be implemented in an in situ deposit of sands.

The cyclic shear stress induced at any point in level ground during an earthquake due to the upward propagation of shear waves can be assessed by means of a simple procedure proposed. If a soil column to a depth z is assumed to move horizontally and if the peak horizontal acceleration on the ground surface is $a_{\max}$, the maximum shear stress $\tau_{\max}$ acting at the bottom of the soil column is given by

$$\tau_{\max} = a_{\max} r_d (\gamma_t)(z/g) \quad (6.3.7a)$$

and,
$$r_d = 1 - 0.015z \quad (6.3.7b)$$

Where γ_t is unit weight of the soil, g is the gravitational acceleration and r_d is a stress reduction coefficient to allow for the deformability of the soil column ($r_d < 1$). It is recommended to use the empirical formula given in Equation (6.3.7b) to compute stress reduction coefficient r_d , where z is in meters. Division of both sides of Equation (6.3.7a) by the effective vertical stress σ'_v gives

$$CSR = \frac{\tau_{\max}}{\sigma'_v} = \frac{a_{\max}}{g} r_d \frac{\sigma_v}{\sigma'_v} \quad (6.3.8)$$

Where, $\sigma_v = \gamma_t z$ is the total vertical stress. Equation (6.3.8) has been used widely to assess the magnitude of shear stress induced in a soil element during an earthquake. One of the advantages of Equation (6.3.8) is that all the vast amount of information on the horizontal accelerations that has ever been recorded on the ground surface can be used directly to assess the shear stress induced by seismic shaking in the horizontal plane within the ground.

The second step is to determine the cyclic resistance ratio (CRR) of the in situ soil. The cyclic resistance ratio represents the liquefaction resistance of the in situ soil. The most commonly used method for determining the liquefaction resistance is to use the data obtained from the standard penetration test. A cyclic triaxial test may also be used to estimate CRR more accurately.

Site Amplification Factor

Site response analysis of a site may be carried out to estimate the site amplification factor. For this purpose, dynamic parameters such as shear modulus and damping factors need to be estimated. The site amplification factor is required to estimate the $a_{\max}$ for a given site properly.

3.9.11 Principles of Structural Design of Foundations

3.9.11.1 Loads and Reactions

3.9.11.1.1 Determination of Loads and Reactions

Footings shall be considered as under the action of downward forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation materials and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces. Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being assumed to carry the computed portion of the total footing load.

3.9.11.1.2 Isolated and Multiple Footing Reactions

When a single isolated footing supports a column, pier or wall, the footing shall be assumed to act as a cantilever element. When footings support more than one column, pier, or wall, the footing slab shall be designed for the actual conditions of continuity and restraint.

3.9.11.1.3 Raft Foundation Reactions

For determining the distribution of contact pressure below a raft it is analysed either as a rigid or flexible foundation considering the rigidity of the raft, and the rigidity of the superstructure and the supporting soil. Consideration shall be given to the increased contact pressure developed along the edges of raft on cohesive soils and the decrease in contact pressure along the edges on granular soils. Any appropriate analytical method reasonably valid for the condition may be used. Choice of a particular method shall be governed by the validity of the assumptions in the particular case. Numerical analysis of rafts using appropriate software may be used for determination of reactions, shears and moments.

Analytical methods (based on beams on elastic foundation) and numerical methods require values of the modulus of subgrade reaction of the soil. For use in preliminary analysis and design, indicative values of the modulus of subgrade reaction for cohesionless soils and cohesive soils are shown in Table 6.3.7 and Table 6.3.8, respectively.

$$k = 0.65 \times \left(\frac{E_s B^4}{EI} \right)^{1/12} \frac{E_s}{(1-\mu^2)} \frac{1}{B} \quad (6.3.9)$$

Table 6.3.7: Modulus of Subgrade Reaction (k) for Cohesionless Soils

Soil Characteristic		*Modulus of Sub-grade Reaction (k) Soil Characteristic (kN/m ³)	
Relative Density	Standard Penetration Test Value (N) (Blows per 300 mm)	For Dry or Moist State	For Submerged State
Loose	<10	15000	9000
Medium	10 to 30	15000 to 47000	9000 to 29000
Dense	30 and over	47000 to 180000	29000 to 108000

*The above values apply to a square plate 300 mm x 300 mm or beams 300 mm wide.

Table 6.3.8 Modulus of Subgrade Reaction (k) for Cohesive Soils

Soil Characteristic		Modulus of Subgrade Reaction, k (kN/m^3)
Consistency	Unconfined Compressive Strength (kN/m^2)	
Stiff	100 to 200	27000
Very Stiff	200 to 400	27000 to 54000
Hard	400 and over	54000 to 108000

* The values apply to a square plate 300 mm x 300 mm. The above values are based on the assumption that the average loading intensity does not exceed half the ultimate bearing capacity.

3.9.11.2 Moment

3.9.11.2.1 Critical Section

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 the entire area of the footing on one side of that vertical plane. The critical section for bending shall be taken at the face of the column, pier, or wall. In the case of columns that are not square or rectangular, the section shall be taken at the side of the concentric square of equivalent area. For footings under masonry walls, the critical section shall be taken halfway between the middle and edge of the wall. For footings under metallic column bases, the critical section shall be taken halfway between the column face and the edge of the metallic base.

3.9.11.2.2 Distribution of Reinforcement

Reinforcement of square footings shall be distributed uniformly across the entire width of footing. Reinforcement of rectangular footings shall be distributed uniformly across the entire width of footing in the long direction. In the short direction, the portion of the total reinforcement given by the following equation shall be distributed uniformly over a band width (centered on center line of column or pier) equal to the length of the short side of the footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)} \quad (6.3.10)$$

Here, β is the ratio of the footing length to width. The remainder of reinforcement required in the short direction shall be distributed uniformly outside the center band width of footing.

3.9.11.2.3 Shear

3.9.11.2.4 Critical Section

Computation of shear in footings, and location of critical section shall be in accordance with relevant sections of the structural design part of the code. Location of critical section shall be measured from the face of column, pier or wall, for footings supporting a column, pier, or wall. For footings supporting a column or pier with metallic base plates, the critical section shall be measured from the location defined in the critical section for moments for footings.

3.9.11.2.5 Critical Section for Footings on Driven Piles/ Bored Piles/ Drilled Piers

Shear on the critical section shall be in accordance with the following. Entire reaction from any driven pile or bored piles, and drilled pier whose center is located $d_p/2$ (d_p = diameter of the pile) or more outside the critical section shall be considered as producing shear on that section. Reaction from any driven pile or drilled shaft whose center is located $d_p/2$ or more inside the critical section shall be considered as producing no shear on that section. For the intermediate position of driven pile or drilled shaft centers, the portion of the driven pile or shaft reaction to be

considered as producing shear on the critical section shall be based on linear interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

3.9.11.3 Reinforcement and Development Length

3.9.11.3.1 Development Length

Computation of development length of reinforcement in footings shall be in accordance with the relevant sections of the structural design part of the code.

3.9.11.3.2 Critical Section

Critical sections for development length of reinforcement shall be assumed at the same locations as defined above as the critical section for moments and at all other vertical planes where changes in section or reinforcement occur.

3.9.11.4 Transfer of Force at Base of Column

3.9.11.4.1 Transfer of Force

All forces and moments applied at base of column or pier shall be transferred to top of footing by bearing on concrete and by reinforcement.

3.9.11.4.2 Lateral Force

Lateral forces shall be transferred to supporting footing in accordance with shear transfer provisions of the relevant sections of the structural design part of the code.

3.9.11.4.3 Bearing Strength of Concrete

Bearing on concrete at contact surface between supporting and supported member shall not exceed concrete bearing strength for either surface.

3.9.11.4.4 Reinforcement

Reinforcement shall be provided across interface between supporting and supported member either by extending main longitudinal reinforcement into footings or by dowels. Reinforcement across interface shall be sufficient to satisfy all of the following:

- (i) Reinforcement shall be provided to transfer all force that exceeds concrete bearing strength in supporting and supported member.
- (ii) If it is required that loading conditions include uplift, total tensile force shall be resisted by reinforcement.
- (iii) Area of reinforcement shall not be less than 0.005 times gross area of supported member, with a minimum of 4 bars.

3.9.11.4.5 Dowel Size

Diameter of dowels, if used, shall not exceed the diameter of longitudinal reinforcements.

3.9.11.4.6 Development Length and Splicing

For transfer of force by reinforcement, development length of reinforcement in supporting and supported member, required splicing shall be in accordance with the relevant sections (Part. 6, Chapter 6) of the structural design part of the code.

3.10 Geotechnical Design of Deep Foundations

3.10.1 Driven Piles

The provisions of this article shall apply to the design of axially and laterally loaded driven piles in soil. Driven pile foundation shall be designed and installed on the basis of a site investigation report that will include subsurface exploration at locations and depths sufficient to determine the position and adequacy of the bearing soil unless adequate data is available upon which the design and installation of the piles can be based. The report shall include:

- (i) Recommended pile type and capacities
- (ii) Driving and installation procedure
- (iii) Field inspection procedure
- (iv) Pile load test, integrity test requirements
- (v) Durability and quality of pile material
- (vi) Designation of bearing stratum or strata

A plan showing clearly the designation of all piles by an identifying system shall be filed prior to installation of such piles. All detailed records for individual piles shall bear an identification corresponding to that shown on the plan. A copy of such plan shall be available at the site for inspection at all times during the construction.

The design and installation of driven pile foundations shall be under the direct supervision of a competent engineer who shall certify that the piles as installed satisfy the design criteria

3.10.1.1 Application

Pile driving may be considered when footings cannot be founded on granular or stiff cohesive soils within a reasonable depth. At locations where soil conditions would normally permit the use of spread footings but the potential for scour exists, piles may be driven as a protection against scour. Piles may also be driven where an unacceptable amount of settlement of spread footings may occur

3.10.1.2 Materials

Driven piles may be cast-in-place concrete, pre-cast concrete, pre-stressed concrete, timber, structural steel sections, steel pipe, or a combination of materials.

3.10.1.3 Penetration

Pile penetration shall be determined based on vertical and lateral load capacities of both the pile and subsurface materials. In general, the design penetration for any pile shall be not less than 3m into hard cohesive or dense granular material, nor less than 6m into soft cohesive or loose granular material.

3.10.1.4 Estimated Pile Length

Estimated pile lengths of driven piles shall be shown on the drawing and shall be based upon careful evaluation of available subsurface information, axial and lateral capacity calculations, and/or past experience.

3.10.1.5 Driven Pile Types

Driven piles shall be classified as "friction" or "end bearing" or a combination of both according to the manner in which load transfer is developed. The ultimate load capacity of a pile consists of two parts. One part is due to friction called skin friction or shaft friction or side shear, and the other is due to end bearing at the base or tip of the pile. If the skin friction is greater than about 80% of the end bearing load capacity, the pile is deemed a friction pile and, if the reverse, an end bearing pile. If the end bearing is neglected, the pile is called a "floating pile".

3.10.1.6 Batter Piles

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, batter piles should be used in the foundation. Where negative skin friction loads are expected, batter piles should be avoided, and an alternate method of providing lateral restraint should be used.

Free standing batter piles are subject to bending moments due to their own weight, or external forces from other sources. Batter piles in loose fill or consolidating deposits may become laterally loaded due to settlement of the surrounding soil. In consolidating clay, special precautions, like provision of permanent casing, shall be taken.

3.10.1.7 Selection of Soil and Rock Properties

Soil and rock properties defining the strength and compressibility characteristics of the foundation materials, are required for driven pile design.

3.10.1.8 Design of Pile Capacity

The design pile capacity is the maximum load that the driven pile shall support with tolerable movement. In determining the design pile capacity the following items shall be considered:

- (i) Ultimate geotechnical capacity (axial and lateral).
- (ii) Structural capacity of pile section (axial and lateral).
- (iii) The allowable axial load on a pile shall be the least value of the above two capacities.

In determining the design axial capacity, consideration shall be given to the following:

- (i) The influence of fluctuations in the elevation of ground water table on capacity.
- (ii) The effects of driving piles on adjacent structure and slopes.
- (iii) The effects of negative skin friction or down loads from consolidating soil and the effects of lift loads from expansive or swelling soils.
- (iv) The influence of construction techniques such as augering or jetting on pile capacity.
- (v) The difference between the supporting capacity single pile and that of a group of piles.
- (vi) The capacity of an underlying strata to support load of the pile group;
- (vii) The possibility of scour and its effect on axial lateral capacity.

3.10.1.9 Ultimate Geotechnical Capacity of Driven Pile for Axial Load

The ultimate load capacity, Q_{ult} , of a pile consists of two parts. One part is due to friction called skin friction or shaft friction or side shear, Q_s and the other is due to end bearing at the base or tip of the pile, Q_b . The ultimate axial capacity (Q_{ult}) of driven piles shall be determined in accordance with the following for compression loading.

$$Q_{ult} = Q_s + Q_b - W \quad (6.3.11)$$

For uplift loading;

$$Q_{ult} \leq 0.7Q_s + W \quad (6.3.12)$$

The allowable or working axial load shall be determined as:

$$Q_{allow} = Q_{ult} / FS \quad (6.3.13)$$

Where, W is the weight of the pile and FS is a gross factor of safety usually greater than 2.5. Often, for compression loading, the weight term is neglected if the weight, W , is considered in estimating imposed loading.

The ultimate bearing capacity (skin friction and/or end bearing) of a single vertical pile may be determined by any of the following methods.

- (i) By the use of static bearing capacity equations
- (ii) By the use of SPT and CPT
- (iii) By field load tests
- (iv) By dynamic methods

3.10.1.10 Static Bearing Capacity Equations for Pile Capacity

The skin friction, Q_s and end bearing Q_b can be calculated as:

$$Q_s = A_s f_s \quad (6.3.14a)$$

and;

$$Q_b = A_b f_b \quad (6.3.14b)$$

Where,

A_s = skin friction area (perimeter area) of the pile = Perimeter $\times$ Length

f_s = skin frictional resistance on unit surface area of pile, that depends on soil properties and loading conditions (drained or undrained)

A_b = end bearing area of the pile = Cross-sectional area of pile tip (bottom)

f_b = end bearing resistance on unit tip area of pile, that depends on soil properties to a depth of $2B$ (B is the diameter for a circular pile section or length of sides for a square pile section) from the pile tip and loading conditions (drained or undrained)

For a layered soil system containing n number of layers, end bearing resistance can be calculated considering soil properties of the layer at which the pile rests, and the skin friction resistance considers all the penetrating layers calculated as:

$$Q_s = \sum_{i=1}^n \Delta Z_i \times (Perimeter)_i \times (f_s)_i \quad (6.3.15)$$

Where, ΔZ_i represents the thickness of any (i th) layer and $(Perimeter)_i$ is the perimeter of the pile in that layer. The manner in which skin friction is transferred to the adjacent soil depends on the soil type. In fine-grained soils, the load transfer is nonlinear and decreases with depth. As a result, elastic compression of the pile is not uniform; more compression occurs on the top part than on the bottom part of the pile. For coarse-grained soils, the load transfer is approximately linear with depth (higher loads at the top and lower loads at the bottom).

In order to mobilize skin friction and end bearing, some movement of the pile is necessary. Field tests have revealed that to mobilize the full skin friction a vertical displacement of 5 to 10 mm is required. The actual vertical displacement depends on the strength of the soil and is independent of the pile length and pile diameter. The full end bearing resistance is mobilized in driven piles when the vertical displacement is about 10% of the pile tip diameter. For bored piles or drilled shafts, a vertical displacement of about 30% of the pile tip diameter is required. The full end bearing resistance is mobilized when slip or failure zones similar to shallow foundations are formed. The end bearing resistance can then be calculated by analogy with shallow foundations. The important bearing capacity factor is N_q .

The full skin friction and full end bearing are not mobilized at the same displacement. The skin friction is mobilized at about one-tenth of the displacement required to mobilize the end bearing resistance. This is important in decid-

ing on the factor of safety to be applied to the ultimate load. Depending on the tolerable settlement, different factors of safety can be applied to skin friction and to end bearing.

Generally, piles driven into loose, coarse-grained soils tend to densify the adjacent soil. When piles are driven into dense, coarse-grained soils, the soil adjacent to the pile becomes loose. Pile driving usually remolds fine-grained soils near the pile shaft. The implication of pile installation is that the intact shear strength of the soil is changed and one must account for this change in estimations of the load capacity.

3.10.1.11 Axial Capacity of Driven Piles in Cohesive Soil using Static Bearing Capacity Equations

The ultimate axial capacity of driven piles in cohesive may be calculated from static formula, given by (6.3.14a), (6.3.14b) and (6.3.15), using a total stress method for undrained loading conditions, or an effective stress method for drained loading conditions. Appropriate values of adhesion factor (α) and coefficient of horizontal soil stress (k_s) for cohesive soils that are consistent with soil condition and pile installation procedure may be used. There are basically two approaches for calculating skin friction:

- (i) **The α -method** that is based on total stress analysis and is normally used to estimate the short term load capacity of piles embedded in fine grained soils. In this method, a coefficient α is used to relate the undrained shear strength c_u or s_u to the adhesive stress (f_s) along the pile shaft. As such,

$$Q_s = \alpha c_u A_s \quad (6.3.16)$$

$$\alpha = 1.0 \quad \text{for clays with } c_u \leq 25 \text{ kN/m}^2$$

$$\alpha = 0.5 \quad \text{for clays with } c_u \geq 70 \text{ kN/m}^2$$

$$\alpha = 1 - \left(\frac{c_u - 25}{70} \right) \quad \text{for clays with } 25 \text{ kN/m}^2 < c_u < 70 \text{ kN/m}^2$$

The end bearing in such a case is found by analogy with shallow foundations and is expressed as:

$$Q_b = (c_u)_b (N_c)_b A_b \quad (6.3.17)$$

N_c is a bearing capacity factor, usually 9. c_u is the undrained shear strength of soil at the base of the pile. The suffix b's are indicatives of base of pile. The general equation for N_c is, however, as follows.

$$N_c = 6 \left[1 + 0.2 \left(\frac{L}{D_b} \right) \right] \leq 9 \quad (6.3.18)$$

The symbol D_b represents the diameter at the base of the pile. The skin friction value, $f_b = (c_u)_b (N_c)_b$ should not exceed 4.0 MPa.

- (ii) **The β -method** is based on an effective stress analysis and is used to determine both the short term and long term pile load capacities. The friction along the pile shaft is found using Coulomb's friction law, where the friction stress is given by $f_s = \mu \sigma'_x = \sigma'_x \tan \phi'$. The lateral effective stress, σ'_x is proportional to vertical effective stress, σ'_z by a co-efficient, K . As such,

$$f_s = K \sigma'_z \tan \phi' = \beta \sigma'_z \quad (6.3.19a)$$

$$\text{Where, } \beta = K \tan \phi' = K_o \tan \phi' = (1 - \sin \phi') \sqrt{OCR} \quad (6.3.19b)$$

ϕ' is the effective angle of internal friction of soil and OCR is the over-consolidation ratio. For normally consolidated clay, β varies from 0.25 to 0.29. The value of β decreases for very long piles, as such a correction factor is used.

$$\text{Correction factor for } \beta = \log \left(\frac{180}{L} \right) \geq 0.5 \quad (6.3.19c)$$

The end bearing capacity is calculated by analogy with the bearing capacity of shallow footings and is determined from:

$$f_b = (\sigma'_v)_b (N_q)_b \quad (6.3.20)$$

Where, N_q is a bearing capacity factor that depends on angle of internal friction ϕ' of the soil at the base of the pile, as presented in Fig. 6.3.2. Subscript “b” designates the parameters at the base soil.

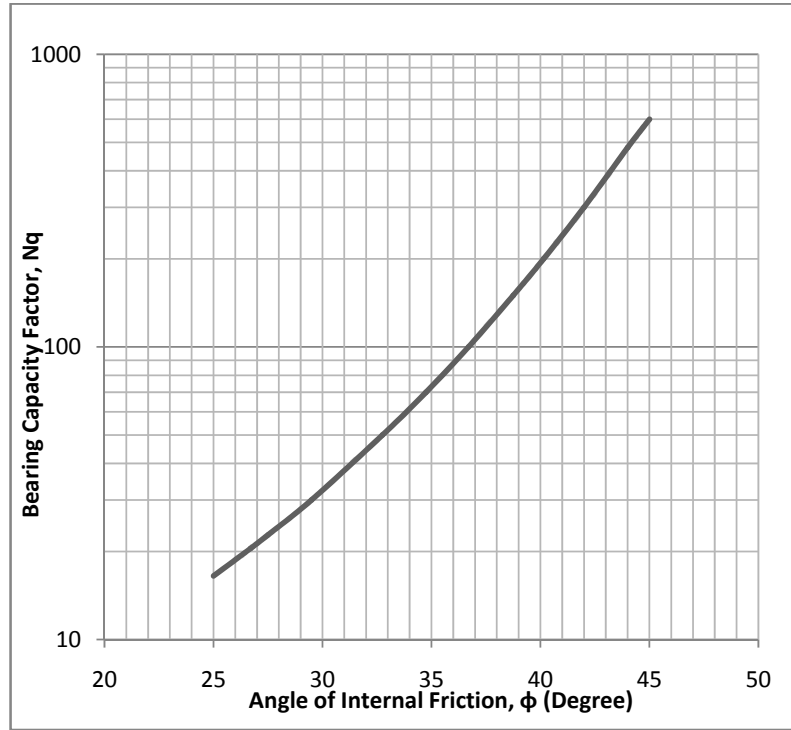


Fig. 6.3.2: Bearing Capacity Factor N_q for Deep Foundation (After Berezantzev, 1961)

3.10.1.12 Axial Capacity of Driven Piles in Cohesive Soil and Non-plastic Silt using SPT Values

Standard Penetration Test N-value is a measure of consistency of clay soil and indirectly the measure of cohesion. The skin friction of pile can thus be estimated from N-value. The following relation may be used for preliminary design of piles in clay and silt soils. The N value used should be corrected for overburden.

For clay and silt: $f_s = 1.67 N \quad (\text{in kPa}) \quad \leq 70 \text{ kPa} \quad (6.3.21)$

For end bearing, the relationship is as under.

For clay: $f_b = 45 N \quad (\text{in kPa}) \quad (6.3.22)$

For silt: $f_b = 40 N \left(\frac{L}{D} \right) \quad (\text{in kPa}) \quad \leq 300 N \text{ and } \leq 10000 \text{ kPa} \quad (6.3.23)$

3.10.1.13 Axial Capacity of Driven Piles in Cohesionless Soil using Static Bearing Capacity Equations

Piles in cohesionless soils shall be designed by effective stress methods of analysis for drained loading conditions. The ultimate axial capacity of piles in cohesionless soils may also be calculated using empirical effective stress

method or from in-situ methods and analysis such as the cone penetration or pressure meter tests. Dynamic formula may be used for driven piles in cohesionless soils such as gravels, coarse sand and deposits where pore pressure developed due to driving is quickly dissipated.

For piles in cohesionless soil, the ultimate side resistance may be estimated using the following formula:

$$f_s = \beta \sigma'_z \quad (6.3.24)$$

Where, σ'_z is the effective vertical stress at the level under consideration. The values for β are as under.

$\beta = 0.10$	for $\phi = 33^\circ$
$\beta = 0.20$	for $\phi = 35^\circ$
$\beta = 0.35$	for $\phi = 37^\circ$

For uncemented calcareous sand the value of β varies from 0.05 to 0.10.

The following equation, as used for cohesive soil, may be used to compute the ultimate end bearing capacity of piles in sandy soil in which, the maximum effective stress, σ'_v allowed for the computation is 240 kPa

$$f_b = (\sigma'_v)_b (N_q)_b \quad (6.3.25)$$

$N_q = 8$ to 12	for loose sand
$N_q = 12$ to 40	for medium sand
$N_q = 40$	for dense sand

Fig. 6.3.2 may also be used to estimate the value of N_q .

Critical Depth for End Bearing and Skin Friction

The vertical effective stress (σ'_v or σ'_z) increases with depth. Hence the skin friction should increase with depth indefinitely. In reality skin friction does not increase indefinitely. It is believed that skin friction would become a constant at a certain depth. This depth is named critical depth. Pile end bearing in sandy soils is also related to effective stress. Experimental data indicates that end bearing capacity does not also increase with depth indefinitely. Due to lack of a valid theory, Engineers use the same critical depth concept adopted for skin friction for end bearing capacity as well. Both the skin friction and the end bearing capacity are assumed to increase till the critical depth, d_c and then maintain a constant value. Following approximations may be used for the critical depth in relation to diameter of pile, D .

$d_c = 10D$	for loose sand
$d_c = 15D$	for medium dense sand
$d_c = 20D$	for dense sand

3.10.1.14 Axial Capacity of Driven Piles in Cohesionless Soil using SPT Values

Standard Penetration Test N-value is a measure of consistency of clay soil and indirectly the measure of cohesion. The skin friction of pile can thus be estimated from N-value. The following relation may be used for clay soils.

$$\text{For large displacement piles: } f_b = 2N \quad (\text{in kPa}) \quad (6.3.26a)$$

$$\text{For large displacement piles: } f_b = 1N \quad (\text{in kPa}) \quad (6.3.26b)$$

For end bearing, the relationship is as under.

$$f_b = 400 N \left(\frac{L}{D} \right) \quad (\text{in kPa}) \leq 400 N \text{ and } \leq 10000 \text{ kPa} \quad (6.3.27)$$

3.10.1.15 Axial Capacity of Driven Pile using Pile Load Test

Generally, the total test load is twice the design load. The pile load test has considered to have failed, if the settlement into the soil, that is the gross settlement minus elastic shortening, is greater than 25 mm at full test load or the settlement into the soil is greater than 13mm, at the end of the test after removal of the load.

3.10.1.16 Selection of Factor of Safety

A factor of safety shall be applied to all estimates of failure load after considering:

- i) The reliability of the value of the ultimate bearing capacity,
- ii) Control of the pile installation procedure
- ii) The type of superstructure and type of loading, and
- iii) Allowable total and differential settlement of the structure.

When ultimate bearing capacity is calculated from either static formula or dynamic formula, the above factors shall be considered. The minimum factor of safety on static formula shall be 3.0. The factor of safety shall actually depend on the reliability of the formula, depending on a particular site and the reliability of the subsoil parameters employed in the calculations. The assumption of a factor of safety shall also consider the load settlement characteristics of the structure as a whole on a given site. The design pile capacity shall be specified on the plans so the factor of safety can be adjusted if the specified construction control procedure is altered. When safe load on a driven pile is assessed by applying a factor of safety to load test data, the minimum safety factor shall be 2.

Settlement is to be limited or differential settlement avoided (i.e., for accurately aligned machinery or a fragile finish of superstructure)

- (i) large impact or vibrating loads are expected
- (ii) soil strength or modulus may be expected to deteriorate with time
- (iii) live load on a structure carried by friction piles is a considerable portion of the total load and approximate the dead load in duration.

The allowable axial load on a pile shall be the least value permitted by consideration of the following factors:

- (i) The capacity of the pile as a structural member.
- (ii) The allowable bearing pressure on soil strata underlying the pile tip.
- (iii) The resistance to penetration of the pile, including resistance to driving, resistance to jacking, the rate of penetration, or other equivalent criteria.
- (iv) The capacity as indicated by load test, where load tests are required.

Driven pile in soil shall be designed for a minimum overall factor of safety of 2.0 against bearing capacity failure (end bearing, side resistance or combined) when the design is based on the results of a load test conducted at the site. Otherwise, it shall be designed for a minimum overall factor of safety 3.0. The minimum recommended overall factor of safety is based on an assumed normal level of field quality control during construction. If a normal level of field quality control cannot be assured, higher minimum factors of safety shall be used. The recommended values of overall factor of safety on ultimate axial load capacity based on specified construction Control is presented in Table 6.3.8.

Partial factor of safety may be used independently for skin friction and end bearing. The values of partial factor of safety may be taken as 1.5 and 3.0 respectively for skin friction and end bearing. The design/allowable load may be taken as the minimum of the values considering overall and partial factor of safety.

Table 6.3.8: Design Factor of Safety for Deep Foundation for Downward and Upward Load

Structure	Design Life (yrs.)	Probability of Failure	Design Factor of Safety			
			Good Control	Normal Control	Poor Control	V. Poor Control
Monument	> 100	10^{-5}	2.30	3.00	3.50	4.00
Permanent	25 -100	10^{-4}	2.00	2.50	2.80	3.40
Temporary	< 25	10^{-3}	1.40	2.00	2.30	2.80
For uplift load, factor of safety is higher. Usually 1.5 to 2.0 times of the values in this chart for downward loading						

3.10.1.17 Group Piles and Group Capacity of Driven Piles

All piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered as being braced, provided that the piles are located in a radial direction from the centroid of the group, not less than 60 degrees apart circumferentially. A two pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Piles supporting walls shall be driven alternately in lines at least 300 mm apart and located symmetrically under the centre of gravity of the wall load, unless effective measures are taken to cater for eccentricity and lateral forces, or the wall piles are adequately braced to provide lateral stability.

Group pile capacity of driven piles should be determined as the product of the group efficiency, number of piles in the group and the capacity of a single pile. In general, a group efficiency value of 1.0 should be used except for friction piles driven in cohesive soils. The minimum center-to-center pile spacing of 2.5B is recommended.

3.10.1.17.1 Pile Caps

Pile caps shall be of reinforced concrete. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of all piles shall be embedded not less than 75 mm into pile caps and the cap shall extend at least 100 mm beyond the edge of all piles. The tops of all piles shall be cut back to sound material before capping. The pile cap shall be rigid enough, so that the imposed load can be distributed on the piles in a group equitably. The cap shall generally be cast over a 75 mm thick levelling course of concrete. The clear cover for the main reinforcement in the cap slab under such condition shall not be less than 60 mm.

3.10.1.18 Lateral Loads (Capacity) on Driven Piles

The design of laterally loaded piles is usually governed by lateral movement criteria. The design of laterally loaded piles shall account for the effects of soil-structure interaction between the pile and ground. Methods of analysis evaluating the ultimate capacity or deflection of laterally loaded piles may be used for preliminary design only as a means to evaluate appropriate pile sections. Lateral capacity of vertical single piles shall be the least of the values calculated on the basis of soil failure, structural capacity of the pile and deflection of the pile head.

Deflection calculations require horizontal subgrade modulus of the surrounding soil. When considering lateral load on piles, the effect of other coexistent loads, including axial load on the pile, shall be taken into consideration for checking structural capacity of the shaft.

For estimating the depth of fixity, established method of analysis shall be used. To determine lateral load capacity, lateral load test to at least twice the proposed design working load shall be made. The resulting allowable load shall not be more than one-half of the test load that produces a gross lateral movement of 25 mm at the ground surface.

Lateral load tests shall be performed. All piles standing unbraced in air, water or soils not capable of providing lateral support shall be designed as columns in accordance with the provisions of this Code.

3.10.1.19 Vertical Ground Movement

The potential for external loading on a pile by vertical ground movements shall be considered as part of the design. Vertical ground movements may result in negative skin friction or downdrag loads due to settlement of compressible soils or may result in uplift loads due to heave of expansive soils. For design purposes, the full magnitude of maximum vertical ground movement shall be assumed.

3.10.1.19.1 Negative Skin Friction (Downward Movement)

Driven piles installed in compressible fill or soft soil subject to compression shall be designed against downward load due to downdrag known as the negative friction of the compressible soil. The potential for external loading on a pile by negative skin friction/downdrag due to settlement of compressible soil shall be considered as a part of the design. Evaluation of negative skin friction shall include a load-transfer method of analysis to determine the neutral point (i.e., point of zero relative displacement) and load distribution along shaft. Due to the possible time dependence associated with vertical ground movement, the analysis shall consider the effect of time on load transfer between the ground and shaft and the analysis shall be performed for the time period relating to the maximum axial load transfer to the pile. If necessary, negative skin friction loads that cause excessive settlement may be reduced by application of bitumen or other viscous coatings to the pile surfaces before In estimating negative skin friction the following factors shall be considered:

- (i) Relative movement between soil and pile shaft.
- (ii) Relative movement between any underlying compressible soil and pile shaft.
- (iii) Elastic compression of the pile under the working load.
- (iv) The rate of consolidation of the compressible layer.

Negative skin friction is mobilized only when tendency for relative movement between pile shaft and surrounding soil exists.

3.10.1.19.2 Expansive Soils (Upward Movement)

Piles driven in swelling soils may be subjected to uplift forces in the zone of seasonal moisture change. Piles shall extend a sufficient distance into moisture-stable soils to provide adequate resistance to swelling uplift forces. In addition, sufficient clearance shall be provided between the ground surface and the underside of pile caps or grade beams to preclude the application of uplift loads at the pile cap. Uplift loads may be reduced by application of bitumen or other viscous coatings to the pile surface in the swelling zone.

3.10.1.20 Dynamic/Seismic Design of Piles

In case of submerged loose sands, vibration caused by earthquake may cause liquefaction or excessive total and differential settlements. This aspect of the problem shall be investigated and appropriate methods of improvements should be adopted to achieve suitable values of N . Alternatively, large diameter drilled pier foundation shall be provided and taken to depths well into the layers which are not likely to liquefy.

3.10.1.21 Protection against Corrosion and Abrasion

Where conditions of exposure warrant, concrete encasement or other corrosion protection shall be used on steel piles and steel shells. Exposed steel piles or steel shells shall not be used in salt or brackish water, and only with caution in fresh water. Details are given in Section 3.7.2.

3.10.1.22 Dynamic Monitoring

Dynamic monitoring may be specified for piles installed in difficult subsurface conditions such as soils with obstructions and boulders to evaluate compliance with structural pile capacity. Dynamic monitoring may also be

considered for geotechnical capacity verification, where the size of the project or other limitations deters static load testing.

3.10.1.23 Maximum Allowable Driving Stresses

Maximum allowable driving stresses in pile material for top driven piles shall not exceed $0.9F_y$ (compression), $0.9F_y$ (tension) for steel piles, $0.85f'_c$ concrete (compression) and $0.7F_y$ (steel reinforcement (tension) for concrete piles and $0.85f'_c - f_{pc}$ (compression) for prestressed concrete piles.

3.10.1.24 Buoyancy

The effects of hydrostatic pressure shall be considered in the design of driven piles, where used with foundation subjected to buoyancy forces.

3.10.1.25 Protection against Deterioration

3.10.1.25.1 Steel Piles

A steel pile foundation design shall consider that steel piles may be subject to corrosion, particularly in fill soils low pH soils (acidic) and marine environments. A field electric resistivity survey or resistivity testing and pH testing of soil and ground water samples should be used to evaluate the corrosion potential. Methods of protecting steel piling in corrosive environments include use of protective coatings, cathodic protection, and increased pile steel area.

3.10.1.25.2 Concrete Piles

A concrete pile foundation design shall consider that deterioration of concrete piles can occur due to sulfates in soil, ground water, or sea water; chlorides in soils and chemical wastes; acidic ground water and organic acids. Laboratory testing of soil and ground water samples for sulfates and pH is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and ground water samples is recommended when chemical wastes are suspected. Methods of protecting concrete piling can include dense impermeable concrete, sulfate resisting portland cement, minimum cover requirements for reinforcing steel, and use of epoxies, resins, or other protective coatings.

3.10.1.25.3 Timber Piles

A timber pile foundation (used for temporary structures) design shall consider that deterioration of timber piles can occur due to decay from wetting and drying cycles or from insects or marine borers. Methods of protecting timber piling include pressure treating with creosote or other wood preservatives.

3.10.1.26 Pile Spacing, Clearance and Embedment

3.10.1.26.1 Pile Spacing

End bearing driven piles shall be proportioned such that the minimum center-to-center pile spacing shall exceed the greater of 750 mm or 2.5 pile diameters/widths. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 100 mm. The spacing of piles shall be such that the average load on the supporting strata will not exceed the safe bearing value of those strata as determined by test boring or other established methods.

Piles deriving their capacity from frictional resistance shall be sufficiently apart to ensure that the zones of soil from which the piles derive their support do not overlap to such an extent that their bearing values are reduced. Generally, in such cases, the spacing shall not be less than 3.0 times the diameter of the shaft.

3.10.1.26.2 Minimum Projection of Pile into the Pile Cap

The tops of piles shall project not less than 100 mm into concrete after all damaged pile material has been removed.

3.10.1.27 Structural Capacity of Driven Pile Section

The cross-section of driven piles shall be of sufficient size and pile material shall have the necessary structural strength to resist all handling stresses during driving or installation and the necessary strength to transmit the load imposed on them to the underlying and surrounding soil. Pile diameter/cross-section of a pile shaft at any level shall not be less than the designated nominal diameter/cross-section. The structural design of piles must consider each of the following loading conditions.

- Handling loads are those imposed on the pile between the time it is fabricated and the time it is in the pile driver leads and ready to be driven. They are generated by cranes, forklifts, and other construction equipment.
- Driving loads are produced by the pile hammer during driving.
- Service loads are the design loads from the completed structures.

The maximum allowable stress on a pile shall not exceed $0.33f'_c$ for precast concrete piles and $33f'_c - f_{pc}$ for prestressed concrete piles and $0.25f_y$ for steel H-piles. The axial carrying capacity of a pile fully embedded in soil with undrained shear strength greater than 10 kN/m^2 shall not be limited by its strength as long column. For weaker soils (undrained shear strength less than 10 kN/m^2), consideration shall be given to determine whether the shaft would behave as a long column. If necessary, suitable reductions shall be made in its structural strength considering buckling. The effective length of a pile not secured against buckling by adequate bracing shall be governed by fixity conditions imposed on it by the structure it supports and by the nature of the soil in which it is installed.

3.10.1.28 Driven Cast-in-Place Concrete Piles

Driven cast-in-place concrete piles shall be in general cast in metal shells driven into the soil that will remain permanently in place. However, other types of cast-in-place piles, plain or reinforced, cased or uncased, may be used if the soil conditions permit their use and if their design and method of placing are satisfactory.

3.10.1.28.1 Shape

Cast-in-place concrete piles may have a uniform cross-section or may be tapered over any portion.

3.10.1.28.2 Minimum Area

The minimum area at the butt of the pile shall be 650 square cms and the minimum diameter at the tip of the pile shall be 200 mm.

3.10.1.28.3 General Reinforcement Requirements

Depending on the driving and installation conditions and the loading condition, the amount of reinforcement and its arrangement shall vary. Cast-in-place piles, carrying axial loads only, where the possibility of lateral forces being applied to the piles is insignificant, need not be reinforced where the soil provides adequate lateral support. Those portions of cast-in-place concrete piles that are not supported laterally shall be designed as reinforced concrete columns and the reinforcing steel shall extend 3000 mm below the plane where the soil provides adequate lateral restraint. Where the shell is smooth pipe and more than 3 mm in thickness, it may be considered as load carrying in the absence of corrosion. Where the shell is corrugated and is at least 2 mm in thickness, it may be considered as providing confinement in the absence of corrosion.

3.10.1.28.4 Reinforcement into Superstructure

Sufficient reinforcement shall be provided at the junction of the pile with the superstructure to make a suitable connection. The embedment of the reinforcement into the cap shall be as **specified for precast piles**

3.10.1.28.5 Shell Requirements

The shell shall be of sufficient thickness and strength, so that it will hold its original form and show no harmful distortion after it and adjacent shells have been driven and the driving core, if any, has been withdrawn. The plans shall stipulate that alternative designs of the shell must be approved by the Engineer before any driving is done.

3.10.1.28.6 Splices

Piles may be spliced provided the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing providing equal results may be considered for approval.

3.10.1.28.7 Reinforcement Cover

The reinforcement shall be placed a clear distance of not less than 50 mm from the cased or uncased sides. When piles are in corrosive or marine environments, or when concrete is placed by the water or slurry displacement methods, the clear distance shall not be less than 75 mm for uncased piles and piles with shells not sufficiently corrosion resistant. Reinforcements shall extend to within 100 mm of the edge of the pile cap. Reinforcements shall extend to within 100 mm of the edge of the pile cap.

3.10.1.28.8 Installation

Steel cased piles shall have the steel shell mandrel driven their full length in contact with surrounding soil, left permanently in place and filled with concrete. No pile shall be driven within 4.5 times the average pile diameter of a pile filled with concrete less than 24 hours old. Concrete shall not be placed in steel shells within the heave range of driving.

3.10.1.28.9 Concreting

For bored or driven cast-in-situ piles, concrete shall be deposited in such a way as to preclude segregation. Concrete shall be deposited continuously until it is brought to the required level. The top surface shall be maintained as level as possible and the formation of seams shall be avoided.

For under-reamed piles, the slump of concrete shall range between 100 mm and 150 mm for concreting in water free holes. For large diameter holes concrete may be placed by tremie or by drop bottom bucket; for small diameter boreholes a tremie shall be utilized.

A slump of 125 mm to 150 mm shall be maintained for concreting by tremie. In case of tremie concreting for piles of smaller diameter and length up to 10 m, the minimum cement content shall be 350 kg/m³ of concrete. For larger diameter and/or deeper piles, the minimum cement content shall be 400 kg/m³ of concrete.

For concreting under water, the concrete shall contain at least 10 per cent more cement than that required for the same mix placed in the dry. The amount of coarse aggregate shall be not less than one and a half times, nor more than two times, that of the fine aggregate. The materials shall be so proportioned as to produce a concrete having a slump of not less than 100 mm, nor more than 150 mm, except where plasticizing admixtures is used in which case, the slump may be 175 mm.

3.10.1.28.10 Structural Integrity

Bored piles shall be installed in such a manner and sequence as to prevent distortion or damage to piles being installed or already in place, to the extent that such distortion or damage affects the structural integrity of the piles.

3.10.1.29 Prestressed Concrete Piles

3.10.1.29.1 Size and Shape

Prestressed concrete piles that are generally octagonal, square or circular shall be of approved size and shape.. Concrete in prestressed piles shall have a minimum compressive strength (cylinder), f'_c of 35 MPa at 28 days. Prestressed concrete piles may be solid or hollow. For hollow piles, precautionary measures should be taken to prevent breakage due to internal water pressure during driving.

3.10.1.29.2 Main Reinforcement

Main reinforcement shall be spaced and stressed so as to provide a compressive stress on the pile after losses; f_{pc} , generally not less than 5 MPa to prevent cracking during handling and installation. Piles shall be designed to resist stresses developed during handling as well as under service load conditions. Bending stresses shall be investigated for all conditions of handling, taking into account the weight of the pile plus 50-percent allowance for impact, with tensile stresses limited to $5\sqrt{f'_c}$.

3.10.1.29.3 Vertical and Spiral Reinforcement

The full length of vertical reinforcement shall be enclosed within spiral reinforcement. For piles up to 600 mm in diameter, spiral wire shall be No.5 (U.S. Steel Wire Gage). Spiral reinforcement at the ends of these piles shall have a pitch of 75 mm for approximately 16 turns.

In addition, the top 150 mm of pile shall have five turns of spiral winding at 25 mm pitch. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 150 mm pitch. For piles having diameters greater than 600 mm. spiral wire shall be No.4 (U.S. Steel Wire Gage). Spiral reinforcement at the end of these piles shall have a pitch of 50 mm for approximately 16 turns. In addition, the top 150 mm of pile shall have four turns of spiral winding at 38 mm pitch. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 100 mm pitch. The reinforcement shall be placed at a clear distance from the face of the prestressed pile of not less than 50 mm.

3.10.1.29.4 Driving and Handling Stresses

A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 28 MPa, but not less than such strength sufficient to withstand handling and driving forces.

3.10.2 Bored Piles

In bored cast in place piles, the holes are first bored with a permanent or temporary casing or by using bentonite slurry to stabilize the sides of the bore. A prefabricated steel cage is then lowered into the hole and concreting is carried by tremie method.

3.10.2.1 Ultimate Geotechnical Capacity of Bored Pile for Axial Load

The basic concept of ultimate bearing capacity and useful equations for axial load capacity are identical to that of driven pile as described in Section 3.11.1.9.

3.10.2.2 Axial Capacity of Bored Piles in Cohesive Soil using Static Bearing Capacity Equations

The ultimate axial capacity of bored piles in cohesive may be calculated from the same static formula as used for driven piles, given by Equations (6.3.14a), (6.3.14b) and (6.3.15), using a total stress method for undrained loading conditions, or an effective stress method for drained loading conditions. The skin friction (f_s) may be taken as 0.67 times the value of driven piles and the end bearing (f_b) may be taken as $1/3^{\text{rd}}$ of driven pile.

3.10.2.3 Axial Capacity of Bored Piles in Cohesive Soil and Non-plastic Silt using SPT Values

The following relation may be used for preliminary design of piles in clay and silt soils. The N value used should be corrected for overburden.

$$\text{For clay and silt: } f_s = 1.11N \quad (\text{in kPa}) \leq 70 \text{ kPa} \quad (6.3.28)$$

For end bearing, the relationship is as under.

$$\text{For clay: } f_b = 15N \quad (\text{in kPa}) \quad (6.3.29)$$

$$\text{For silt: } f_b = 100N \quad (\text{in kPa}) \quad (6.3.30)$$

3.10.2.4 Axial Capacity of Bored Piles in Cohesionless Soil using Static Bearing Capacity Equations

The ultimate axial capacity of bored piles in cohesive may be calculated from the same static formula as used for driven piles described section 3.1.11.13. The skin friction (f_s) may be taken as 0.67 times the value of driven piles and the end bearing (f_b) may be taken as $1/3^{\text{rd}}$ of driven pile.

Critical Depth for End Bearing and Skin Friction

Similar to driven piles, following approximations may be used for the critical depth in relation to diameter of pile, D.

$d_c = 10D$	for loose sand
$d_c = 15D$	for medium dense sand
$d_c = 20D$	for dense sand

3.10.2.5 Axial Capacity of Bored Piles in Cohesionless Soil using SPT Values

Standard Penetration Test N-value is a measure of consistency of clay soil and indirectly the measure of cohesion. The skin friction of pile can thus be estimated from N-value. The following relation may be used for clay soils.

$$f_s = 1.33N \quad (\text{in kPa}) \leq 60 \text{ kPa} \quad (6.3.31)$$

For end bearing, the relationship is as under.

$$f_b = 133N \left(\frac{L}{D} \right) \quad (\text{in kPa}) \leq 133N \text{ and } \leq 10000 \text{ kPa} \quad (6.3.32)$$

3.10.2.6 Axial Capacity of Bored Pile using Pile Load Test

The procedures and principles of pile load test for ultimate capacity are similar to that of driven piles.

3.10.2.7 Selection of Factor of Safety

Selection of factor of safety for axial capacity of bored pile is similar to that used for driven piles.

3.10.2.8 Group Capacity of Bored Piles and other

The behavior of group bored piles is almost similar to that of driven piles. For the pile cap, lateral load capacity, vertical ground movement, negative skin friction, piles in expansive soil, dynamic and seismic design, corrosion protection, dynamic monitoring and buoyancy, Sections 3.11.1.17 should be consulted as they are similar for both driven and bored piles.

3.10.3 Settlement of Driven and Bored Piles

The settlement of axially loaded piles and pile groups at the allowable loads shall be estimated. Elastic analysis, load transfer and/or finite element techniques may be used. The settlement of the pile or pile group shall not exceed the tolerable movement limits as recommended for shallow foundations (Table 6.3.7).

When a pile is loaded two things would happen involving settlement.

- The pile would settle into the soil
- The pile material would compress due to load

The settlement of a single pile can be broken down into three distinct parts.

- Settlement due to axial deformation, S_{ax}
- Settlement at the pile tip, S_{pt}
- Settlement due to skin friction, S_{sf}

Thus ,
$$S_{t(single)} = S_{ax} + S_{pt} + S_{sf} \quad (6.3.33)$$

Moreover, piles acting in a group could undergo long term consolidation settlement.

Settlement due to axial deformation of a single pile can be estimated as:

$$S_{ax} = \frac{(Q_p + aQ_s)L}{AE_p} \quad (6.3.34)$$

Where,

Q_p =	Load transferred to the soil at tip level
Q_s =	Total skin friction load
L =	Length of the pile
A =	Cross section area of the pile
E_p =	Young's modulus of pile material
a =	0.5 for clay and silt soils
a =	0.67 for sandy soil

Pile tip settlement, S_{tp} can be estimated as:

$$S_{tp} = \frac{c_p Q_p}{D q_o} \quad (6.3.35)$$

Where,

Q_p =	Load transferred to the soil at tip level
D =	Diameter of the pile
q_o =	Ultimate end bearing capacity
C_p =	Empirical coefficient as given in Table 6.3.9
a =	0.67 for sandy soil

Table 6.3.9: Typical Values of C_p for Settlement Calculation of Single Pile

Soil Type	Values of C_p	
	Driven Pile	Bored Pile
Dense Sand	0.02	0.09
Loose Sand	0.04	0.18
Stiff Clay	0.02	0.03
Soft Clay	0.03	0.06
Dense Silt	0.03	0.09
Loose Silt	0.05	0.12

Skin friction acting along the shaft would stress the surrounding soil. Skin friction acts upward direction along the pile. The force due to pile on surrounding soil would be in downward direction. When the pile is loaded, the pile would slightly move down. The pile would drag the surrounding soil with it. Hence, the pile settlement would occur due to skin friction as given by:

$$S_{sf} = \frac{C_s Q_s}{D q_o} \quad (6.3.36)$$

Where,

C_s = Empirical coefficient = $\left(0.93 + 0.16 \frac{L}{D}\right) C_p$

C_p = Empirical coefficient as given in Table 6.3.9

Q_s = Total skin friction load

D = Diameter of the pile

q_o = Ultimate end bearing capacity

Short Term Pile Group Settlement

Short term or elastic pile group settlement can be estimated using the following relation.

$$S_g = S_{t(single)} \left(\frac{B}{D}\right)^{0.5} \quad (6.3.37)$$

Where,

S_g = Settlement of the pile group

$S_{t(single)}$ = Total settlement of a single pile

B = Smallest dimension of the pile group

D = Diameter of the pile

Interestingly, geometry of the group does not have much of an influence on the settlement. As such, Group Settlement Ratio, R_s of a pile group consisting of n number of piles can be approximated as follows.

$$R_s = \frac{S_g}{S_{t(single)}} = (n)^{0.5} \quad (6.3.38)$$

Settlement of the group can be estimated as the highest value as obtained from Equations (6.3.37) and (6.3.38).

Long Term Settlement for Pile Group

For pile groups, settlement due to consolidation is more important than for single piles. Consolidation settlement of pile group in clay soil is computed using the following simplified assumptions.

- The pile group is assumed to be a solid foundation with a depth $2/3^{\text{rd}}$ the length of the piles
- Effective stress at mid-point of the clay layer is used to compute settlement

If soil properties are available consolidation settlement may be obtained from the following equation. The depth of significant stress increase (10%) or the depth of bed rock whichever is less should be taken for computation of settlement. Stress distribution may be considered as 2 vertical to 1 horizontal.

$$\text{Consolidation settlement, } S = \frac{C_c H}{1+e_0} \log \frac{\sigma'_o + \sigma'_p}{\sigma'_o} \quad (6.3.39)$$

Where,

- C_c = Compression index of soil
- e_0 = initial void ratio
- H = Thickness of the clay layer
- σ'_o = Initial effective stress at mid point of the clay layer
- σ'_p = Increase in effective stress at mid point of the clay layer due to pile load.

In absence of soil properties the following empirical equations may be used to estimate the long term (consolidation) settlement of clay soils.

For clay:
$$S = \frac{H}{M} \text{Ln} \left(\frac{\sigma'_1}{\sigma'_o} \right)^j \quad (6.3.40)$$

For sand:
$$S = \frac{2H}{M} \left[\left(\frac{\sigma'_1}{\sigma'_r} \right)^j - \left(\frac{\sigma'_o}{\sigma'_r} \right)^j \right] \quad (6.3.41)$$

Where,

- H = Thickness of the clay layer
- σ'_o = Initial effective stress at mid point of the clay layer
- σ'_1 = New effective stress at mid point of the clay layer after pile load.
- σ'_r = Reference stress (100 kPa)
- M = Dimensionless modulus number as obtained from Table 6.3.10
- j = Stress exponent as obtained from Table 6.3.10.

Table 6.3.10: Settlement Parameters

Soil	Density	Modulus Number, M	Stress Exponent, j
Till	V. Dense to Dense	1000 - 300	1.0
Gravel	-	400 - 40	0.5
Sand	Dense	400 - 250	0.5
Sand	Medium Dense	250 - 150	0.5
Sand	Loose	150 - 100	0.5
Silt	Dense	200 - 80	0.5
Silt	Medium Dense	80 - 60	0.5
Silt	Loose	60 - 40	0.5
Silty Clay	Stiff	60 - 40	0.5
Silty Clay	Medium Stiff	20 - 10	0.5
Silty Clay	Soft	10 - 5	0.5
Marine Clay	Soft	20 - 5	0.0
Organic Clay	Soft	20 - 5	0.0
Peat	-	5 - 1	0.0

3.10.4 Drilled Shafts/ Drilled Piers

3.10.4.1 General

Large diameter (more than 400 mm) bored piles are sometimes classified as drilled shaft or drilled piers. They are usually provided with enlarged base called bell. The provisions of this article shall apply to the design of axially and laterally loaded drilled shafts/ drilled piers in soil or extending through soil to or into rock.

3.10.4.2 Application

Drilled shafts may be considered when spread footings cannot be founded on suitable soil within a reasonable depth and when piles are not economically viable due to high loads or obstructions to driving. Drilled shafts may be used in lieu of spread footings as a protection against scour. Drilled shafts may also be considered to resist high lateral or uplift loads when deformation tolerances are small.

3.10.4.3 Material

Shafts shall be cast-in-place concrete and may include deformed bar steel reinforcement, structural steel sections, and/or permanent steel casing as required by design.

3.10.4.4 Embedment

Shaft embedment shall be determined based on vertical and lateral load capacities of both the shaft and sub-surface materials.

3.10.4.5 Batter Shafts

The use of battered shafts to increase the lateral capacity of foundations is not recommended due to their difficulty of construction and high cost. Instead, consideration should first be given to increasing the shaft diameter to obtain the required lateral capacity.

3.10.4.6 Selection of Soil Properties

Soil and rock properties defining the strength and compressibility characteristics of the foundation materials are required for drilled shaft design.

3.10.4.7 Geotechnical Design

Drilled shafts shall be designed to support the design loads with adequate bearing and structural capacity, and with tolerable settlements. In addition, the response of drilled shafts subjected to seismic and dynamic loads shall be evaluated.

Shaft design shall be based on working stress principles using maximum un-factored loads derived from calculations of dead and live loads from superstructures, substructures, earth (i.e., sloping ground), wind and traffic. Allowable axial and lateral loads may be determined by separate methods of analysis.

The design methods presented herein for determining axial load capacity assume drilled shafts of uniform cross section, with vertical alignment, concentric axial loading, and a relatively horizontal ground surface. The effects of an enlarged base, group action, and sloping ground are treated separately

3.10.4.7.1 Bearing Capacity Equations for Drilled Shaft

The ultimate axial capacity (Q_{ult}) of drilled shafts shall be determined in accordance with the principles laid for bored piles.

Cohesive Soil

Skin friction resistance in cohesive soil may be determined using either the α -method or the β -method as described in the relevant section of driven piles. However, for clay soil, α -method has wide been used by the engineers. This method gives:

$$f_s = \alpha s_u \quad (6.3.42)$$

Where, f_s = Skin friction
 s_u = undrained shear strength of soil along the shaft

$\alpha =$ adhesion factor = 0.55 for undrained shear strength ≤ 190 kPa (4000 psf)

For higher values of s_u the value of α may be taken from Fig. 6.3.3 as obtained from test data of previous investigators. The skin friction resistance should be ignored in the upper 1.5 m of the shaft and along the bottom one diameter of straight shafts because of interaction with the end bearing. If end bearing is ignored for some reasons, the skin friction along the bottom one diameter may be considered. For belled shaft, skin friction along the surface of the bell and along the shaft for a distance of one shaft diameter above the top of bell should be ignored.

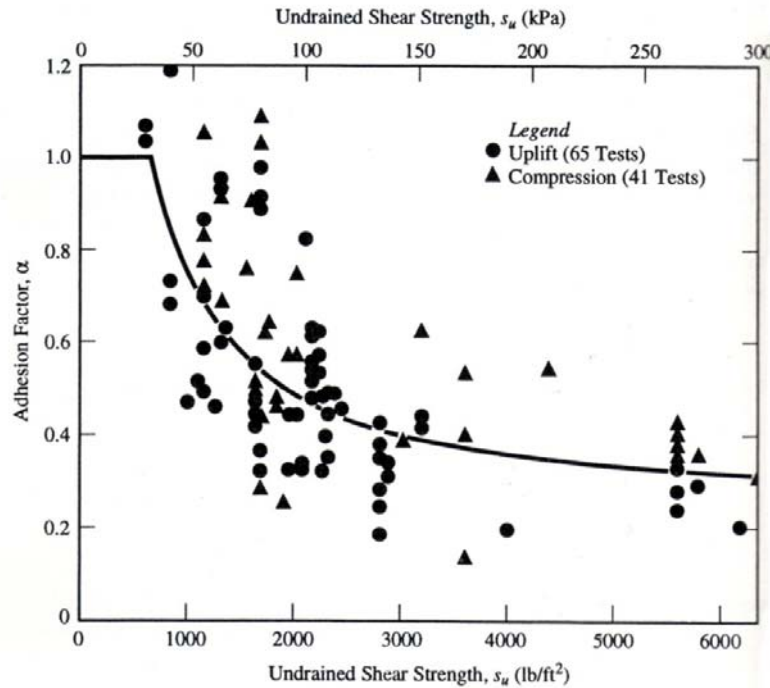


Fig. 6.3.3 Adhesion Factor α for Drilled Shaft (after Coduto, 1994)

For end bearing of cohesive soil, the following relations given by Equations (6.3.43) and (6.3.44) are recommended.

$$f_b = N_c s_u \leq 4000 \text{ kPa} \quad (6.3.43)$$

$$N_c = 6 \left[1 + 0.2 \left(\frac{L}{D_b} \right) \right] \leq 9 \quad (6.3.18)$$

Where,
 f_b = End bearing stress
 s_u = undrained shear strength of soil along the shaft
 N_c = Bearing capacity factor
 L = Length of the pile (Depth to the bottom of the shaft)
 D_b = Diameter of the shaft base

If the base diameter is more than 1900 mm, the value of f_b from Equation (6.3.43) could produce settlements greater than 25 mm, which would be unacceptable for most buildings. To keep settlement within tolerable limits, the value of f_b should be reduced to f'_b by multiplying a factor F_r such that:

$$f'_b = F_r f_b \quad (6.3.44a)$$

$$F_r = \frac{2.5}{120 \omega_1 D_b / B_r + \omega_2} \leq 1.0 \quad (6.3.44b)$$

$$\omega_1 = 0.0071 + 0.0021 \left(\frac{L}{D_b} \right) \leq 0.0015 \quad (6.3.44c)$$

$$\omega_2 = 1.59 \sqrt{\frac{s_u}{\sigma_r}} \quad 0.5 \leq \omega_2 \leq 1.5 \quad (6.3.44d)$$

Where, B_r = Reference width = 1 ft = 0.3 m = 12 inch = 300 mm
 σ_r = Reference stress = 100 kPa = 2000 psf

Cohesionless Soil

Skin friction resistance in cohesionless soil is usually determined using the β -method. The relevant equation is reproduced again:

$$f_s = \beta \sigma'_z \quad (6.3.45)$$

$$\beta = K \tan \phi_s \quad (6.3.46)$$

Where, f_s = Skin friction
 σ'_z = Effective vertical stress at mid point of soil layer
 K = Coefficient of lateral earth pressure
 ϕ_s = Soil shaft interface friction angle

The values of K and ϕ_s can be obtained from the chart of Tables 6.3.11, from the soil friction angle, ϕ and preconstruction coefficient of lateral earth pressure K_o . However, K_o is very difficult to determine. An alternative is to compute β directly using the following empirical relation.

$$\beta = 1.5 - 0.135 \sqrt{\frac{z}{B_r}} \quad (6.3.47)$$

Where, B_r = Reference width = 1 ft = 0.3 m = 12 inch = 300 mm
 z = Depth from the ground surface to the mid point of the strata

Table 6.3.11: Typical ϕ_s/ϕ and K/K_o Values for the Design of Drilled Shaft

Construction Method	ϕ_s/ϕ	Construction Method	K/K_o
Open hole or temporary casing	1.0	Dry construction with minimal side wall disturbance and prompt concreting	1
Slurry method – minimal slurry cake	1.0	Slurry construction – good workmanship	1
Slurry method – heavy slurry cake	0.8	Slurry construction – poor workmanship	2/3
Permanent casing	0.7	Casing under water	5/6

The unit end bearing capacity for drilled shaft in cohesionless soils will be less than that for driven piles because of various reasons like soil disturbance during augering, temporary stress relief while the hole is open, larger diameter and depth of influence etc. The reasons are not well defined, as such the following empirical formula developed by Reese and O'Neill (1989) may be suggested to use to estimate end bearing stress.

$$f_b = 0.60 \sigma_r N \leq 4500 \text{ kPa} \quad (6.3.48)$$

Where, f_b = Unit bearing resistance
 σ'_r = Reference stress = 100 kPa = 2000 psf
 N = Mean SPT value for the soil between the base of the shaft and a depth equal to two times the base diameter below the base. No overburden correction is required ($N = N_{60}$)

If the base diameter is more than 1200 mm, the value of f_b from Equation (6.3.48) could produce settlements greater than 25 mm, which would be unacceptable for most buildings. To keep settlement within tolerable limits, the value of f_b should be reduced to f'_b by multiplying a factor F_r such that:

$$f'_b = F_r f_b \quad (6.3.49a)$$

$$F_r = 4.17 \frac{B_r}{D_b} \leq 1.0 \quad (6.3.49b)$$

Where, B_r = Reference width=1 ft = 0.3 m = 12 inch = 300 mm
 D_b = Base diameter of drilled shaft

3.10.4.7.2 Other Methods of Evaluating Axial Load Capacity

A number of other methods are available to estimate the ultimate axial load capacity of drilled shafts. These methods are based on N-values obtained from Standard Penetration Test (SPT) and on angle of internal friction of sand. These methods may also be used to estimate the ultimate load carrying capacity of drilled shafts. Three of these methods are as follows:

- (i) Method based on the Standard Penetration Test (Canadian Foundation Engineering Manual, 1985)
- (ii) Method based on Theory of Plasticity (Canadian Foundation Engineering Manual, 1985)
- (iii) Tomlinson (1995) Method

These methods are summarized in **Appendix 6.3.C**.

3.10.4.7.3 Factor of Safety

Similar to bored and driven piles, drilled shafts shall be designed for a minimum overall factor of safety of 2.0 against bearing capacity failure (end bearing, side resistance or combined) when the design is based on the results of a load test conducted at the site. Otherwise, it shall be designed for a minimum overall factor of safety 3.0. The minimum recommended overall factor of safety is based on an assumed normal level of field quality control during construction. If a normal level of field quality control cannot be assured, higher minimum factors of safety shall be used. The recommended values of overall factor of safety on ultimate axial load capacity based on specified construction Control is presented in Table 6.3.8.

3.10.4.7.4 Deformation and Settlement of Axially Load Drilled Shaft

Similar to driven and bored piles, settlement of axially loaded shafts at working or allowable loads shall be estimated using elastic or load transfer analysis methods. For most cases, elastic analysis will be applicable for design provided the stress levels in the shaft are moderate relative to Q_{ult} . Analytical methods are similar to that provided in Section 3.11.3 for driven and bored piles. The charts provided in **Appendix 6.3.C** may also be used to estimate the settlement of drilled shaft.

3.10.4.7.5 Layered Soil Profile

The short-term settlement of shafts in a layered soil profile may be estimated by summing the proportional settlement components from layers of cohesive and cohesionless soil comprising the subsurface profile

3.10.4.7.6 Tolerable Movement

Tolerable axial displacement criteria for drilled shaft foundations shall be developed by the structural designer consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of un-acceptable displacements on the structure performance. Drilled shaft displacement analyses shall be based on the results of in-situ and/or laboratory testing to characterize the load-deformation behavior of the foundation materials.

3.10.4.8 Group Loading of Drilled Shaft

3.10.4.8.1 Cohesive Soil

Evaluation of group capacity of shafts in cohesive soil shall consider the presence and contact of a cap with the ground surface and the spacing between adjacent shafts.

For a shaft group with a cap in firm contact with the ground, Q_{ult} may be computed as the lesser of (1) the sum of the individual capacities of each shaft in the group or (2) the capacity of an equivalent pier defined in the perimeter area of the group. For the equivalent pier, the shear strength of soil shall not be reduced by any factor (e.g., α_1) to determine the Q_s component of Q_{ult} , the total base area of the equivalent pier shall be used to determine the Q_T component of Q_{ult} and the additional capacity of the cap shall be ignored.

If the cap is not in firm contact with the ground, or if the soil at the surface is loose or soft, the individual capacity of each shaft should be reduced to ζ times Q_T for an isolated shaft, where $\zeta = 0.67$ for a center-to-center (CTC) spacing of $3B$ (where B is the shaft diameter) and $\zeta = 1.0$ for a CTC spacing of $6B$. For intermediate spacings, the value of ζ may be determined by linear interpolation. The group capacity may then be computed as the lesser of (1) the sum of the modified individual capacities of each shaft in the group, or (2) the capacity of an equivalent pier as described above.

3.10.4.8.2 Cohesionless Soil

Evaluation of group capacity of shafts in cohesion soil shall consider the spacing between adjacent shafts. Regardless of cap contact with the ground, the individual capacity of each shaft should be reduced to times Q_T for an isolated shaft, where $\zeta = 0.67$ for a center-to-center (CTC) spacing of $3B$ and $\zeta = 1.0$ for a CTC spacing of $8B$. For intermediate spacings, the value of ζ may be determined by linear interpolation. The group capacity may be computed as the lesser of (1) the sum of the modified individual capacities of each shaft in the group or (2) the capacity of an equivalent pier circumscribing the group including resistance over the entire perimeter and base areas.

3.10.4.8.3 Strong Soil Overlying Weak Soil

If a group of shafts is embedded in a strong soil deposit which overlies a weaker deposit (cohesionless and cohesive soil), consideration shall be given to the potential for a punching failure of the tip into the weaker soil strata. For this case, the unit tip capacity of the equivalent shaft (q_E) may be determined using the following:

$$q_E = \frac{HB_t}{10} (q_{UP} - q_{Lo}) \leq q_{UP} \quad (6.3.50)$$

In the above equation q_{UP} is the ultimate unit capacity of an equivalent shaft bearing in the stronger upper layer and q_{Lo} is the ultimate unit capacity of an equivalent shaft bearing in the weaker underlying soil layer. If the underlying soil unit is a weaker cohesive soil strata, careful consideration shall be given to the potential for large settlements in the weaker layer.

3.10.4.9 Lateral Loads on Drilled Shaft

3.10.4.9.1 Soil Layering

The design of laterally loaded drilled shafts in layered soils shall be based on evaluation of the soil parameters characteristic of the respective layers

3.10.4.9.2 Ground Water

The highest anticipated water level shall be used for design

3.10.4.9.3 Scour

The potential for loss of lateral capacity due to scour shall be considered in the design. If heavy scour is expected, consideration shall be given to designing the portion of the shaft that would be exposed as a column. In all cases, the shaft length shall be determined such that the design structural load can be safely supported entirely below the probable scour depth.

3.10.4.9.4 Group Action

There is no reliable rational method for evaluating the group action for closely spaced, laterally loaded shafts. Therefore, as a general guide, drilled shaft with diameter B in a group may be considered to act individually when the center-to-center (CTC) spacing is greater than $2.5B$ in the direction normal to loading, and $CTC > 8B$ in the direction parallel to loading. For shaft layout not conforming to these criteria, the effects of shaft interaction shall be considered in the design. As a general guide, the effects of group action for in-line $CTC < 8B$ may be considered using the ratios (CGS, 1985) appearing as below:

Centre to Centre Shaft Spacing for In-line Loading	Ratio of Lateral Resistance of Shaft in Group to Single Shaft
8B	1.00
6B	0.70
4B	0.40
3B	0.25

3.10.4.9.5 Cyclic Loading

The effects of traffic, wind, and other non-seismic cyclic loading on the load-deformation behavior of laterally loaded drilled shafts shall be considered during design. Analysis of drilled shafts subjected to cyclic loading may be considered in the COM624 analysis (Reese, 1984).

3.10.4.9.6 Combined Axial and Lateral Loading

The effects of lateral loading in combination with axial loading shall be considered in the design. Analysis of drilled shafts subjected to combined loading may be considered in the COM624 analysis (Reese, 1984).

3.10.4.9.7 Sloping Ground

For drilled shafts which extend through or below sloping ground, the potential for additional lateral loading shall be considered in the design. The general method of analysis developed by Borden and Gabr (1987) may be used for the analysis of shafts in stable slopes. For shafts in marginally stable slopes, additional consideration should be given for smaller factors of safety against slope failure or slopes showing ground creep, or when shafts extend through fills overlying soft foundation soils and bear into more competent underlying soil or rock formations. For unstable ground, detailed explorations, testing and analysis are required to evaluate potential additional lateral loads due to slope movements.

3.10.4.9.8 Tolerable Lateral Movements

Tolerable lateral displacement criteria for drilled shaft foundations shall be developed by the structural designer consistent with the function and type of structure, fixity, anticipated service life, and consequences of unacceptable displacements on the structure performance. Drilled shaft lateral displacement analysis shall be based on the results of in-situ and/or laboratory testing to characterize the load-deformation behavior of the foundation materials.

3.10.4.9.9 Uplift Loads on Drilled Shaft

Uplift capacity shall rely only on side resistance in conformance with related articles for driven piles. If the shaft has an enlarged base, Q_s shall be determined in conformance with related articles for driven piles.

3.10.4.9.10 Consideration of Vertical Ground Movement

The potential for external loading on a shaft by vertical ground movement (i.e., negative skin friction down-drag due to settlement of compressible soil or uplift due to heave of expansive soil) shall be considered as a part of design. For design purposes, it shall be assumed that the full magnitude of maximum potential vertical ground movement occurs.

Negative Skin Friction

Evaluation of negative skin friction shall include a load-transfer method of analysis to determine the neutral point (i.e., point of zero relative displacement) and load distribution along shaft (e.g., Reese and O'Neill, 1988). Due to the possible time dependence associated with vertical ground movement, the analysis shall consider the effect of time on load transfer between the ground and shaft and the analysis shall be performed for the time period relating to the maximum axial load transfer to the shaft. Evaluation of negative skin friction shall include a load-transfer method of analysis to determine the neutral point (i.e., point of zero relative displacement) and load distribution along shaft (e.g., Reese and O'Neill, 1988). Due to the possible time dependence associated with vertical ground movement, the analysis shall consider the effect of time on load transfer between the ground and shaft and the analysis shall be performed for the time period relating to the maximum axial load transfer to the shaft..

Expansive Soils

Shafts designed for and constructed in expansive soil shall extend to a sufficient depth into moisture-stable soils to provide adequate anchorage to resist uplift movement. In addition; sufficient clearance shall be provided between the ground surface and underside of caps or beams connecting shafts to preclude the application of uplift loads at the shaft/cap connection from swelling ground conditions.

3.10.4.9.11 Dynamic/Seismic Design of Drilled Shaft

Refer to Seismic Design section of this code and Lam and Martin (1986a; 1986b) for guidance regarding the design of drilled shafts subjected to dynamic and seismic loads.

3.10.4.10 Structural Design, Shaft Dimensions and Shaft Spacing

3.10.4.10.1 Design

Drilled shafts shall be designed to resist failure loads to insure that the shaft will not collapse or suffer loss of serviceability due to excessive stress and/or deformation.

3.10.4.10.2 Dimensions

All shafts should be sized in 150 mm increments with a minimum shaft diameter of 450 mm. The diameter of columns supported by shafts shall be less than or equal to the shaft diameter B .

3.10.4.10.3 Center to Center Spacing

The center-to-center spacing of drilled shafts of diameter B should be $3B$ or greater to avoid interference between adjacent shafts during construction. If closer spacing is required, the sequence of construction shall be specified and the interaction effects between adjacent shafts shall be evaluated by the designer.

3.10.4.10.4 Reinforcement Spacing, Clearance and Embedment

Reinforcement

Where the potential for lateral loading is insignificant, drilled shafts need to be reinforced for axial loads only. Those portions of drilled shafts that are not supported laterally shall be designed as reinforced concrete columns in accordance with relevant sections in structural design part of the code and the reinforcing steel shall extend a minimum of 5 m below the plane where the soil provides adequate lateral restraint. Where permanent steel casing is used and the shell is smooth pipe and more than 3 mm in thickness, it may be considered as load carrying in the absence of corrosion.

The design of longitudinal and spiral reinforcement shall be in conformance with the requirements of the relevant sections of the structural design part of the code. Development of length of deformed reinforcement shall be in conformance with the relevant sections of the structural design part of the code.

Longitudinal Bar Spacing

The minimum clear distance between longitudinal reinforcement shall not be less than 3 times the bar diameter nor 3 times the maximum aggregate size. If bars are bundled in forming the reinforcing cage, the minimum clear distance between longitudinal reinforcement shall not be less than 3 times the diameter of the bundled bars. Where heavy reinforcement is required, consideration may be given to an inner and outer reinforcing cage.

Splices

Splices shall develop the full capacity of the bar in tension and compression. The location of splices shall be staggered around the perimeter of the reinforcing cage so as not to occur at the same horizontal plane. Splices may be developed by lapping, welding, and special approved connectors. Splices shall be in conformance with the relevant sections of the structural design part of the code.

Transverse Reinforcement

Transverse reinforcement shall be designed to resist stresses caused by fresh concrete flowing from inside the cage to the side of the excavated hole. Transverse reinforcement may be constructed of hoops or spiral steel.

Handling Stresses

Reinforcement cages shall be designed to resist handling and placement stresses.

Reinforcement Cover

The reinforcement shall be placed a clear distance of not less than 50 mm from the permanently cased or 75 mm from the uncased sides. When shafts are constructed in corrosive or marine environments, or when concrete is placed by the water or slurry displacement methods, the clear distance shall not be less than 100 mm for uncased shafts and shafts with permanent casings not sufficiently corrosion resistant.

The reinforcement cage shall be centered in the hole using centering devices. All steel centering devices shall be epoxy coated.

Reinforcement into Superstructure

Sufficient reinforcement shall be provided at the junction of the shaft with the superstructure to make a suitable connection. The embedment of the reinforcement into the cap shall be in conformance with relevant articles of the structural design part of the code.

3.10.4.10.5 Enlarged Bases

Enlarged bases shall be designed to insure that plain concrete is not overstressed. The enlarged base shall slope at a side angle not less than 30 degrees from the vertical and have a bottom diameter not greater than 3 times diameter of the shaft. The thickness of the bottom edge of enlarged base shall not be less than 150 mm.

3.10.4.11 Construction and Concreting of Drilled Shafts

3.10.4.11.1 Method of Construction

Drilled shafts may be constructed using the dry, casing, or wet method of construction, or a combination of methods. In every case, excavation of hole, placement of concrete, and all other aspects of shaft construction shall be performed in conformance with the provisions of this code.

The load capacity and deformation behavior of drilled shafts can be greatly affected by the quality and methods of construction. The effects of construction methods are incorporated in design by application of factor of safety consistent with the expected construction methods and level of field quality control measures undertaken as described in the relevant sections for driven piles.

Where the spacing between shafts in a group is restricted, consideration shall be given to the sequence of construction to minimize the effect of adjacent shaft construction operations on recently constructed shafts.

The following construction procedure shall be followed:

- (i) Place permanent/temporary steel casing in position and embed casing toe into firm strata.
- (ii) Bore and excavate inside the steel casing down to casing toe level, or to a level approved, and continue excavation to final pile tip level using drilling mud. The fluid level inside casings shall at all times be at least 2 metres higher than outside the casings.
- (iii) Carefully clean up all mud or sedimentation from the bottom of borehole.
- (iv) Place reinforcement cage, inspection pipes etc.
- (v) Concrete continuously under water, or drilling fluid, by use of the tremie method.
- (vi) After hardening, break out the top section of the concrete pile to reach sound concrete.

In drilling of holes for all piles, bentonite and any other material shall be mixed thoroughly with clean water to make a suspension which shall maintain the stability of the pile excavation for the period necessary to place concrete and complete construction. The control tests shall cover the determination of density, viscosity, gel strength and pH values. Bentonite slurry shall meet the Specifications as shown in Table 6.3.12.

Table 6.3.12 Specifications of Bentonite Slurry

Item to be Measured	Range of Results at 20° C	Test Method
Density during drilling to support excavation	greater than 1.05 g/ml	Mud density Balance
Density prior to concreting	less than 1.25 g/ml	Mud density Balance
Viscosity	30 - 90 seconds	Marsh Cone Method
pH	9.5 to 12	pH indicator paper strips or electrical pH meter

Temporary casing of approved quality or an approved alternative method shall be used to maintain the stability of pile excavations, which might otherwise collapse. Temporary casings shall be free from significant distortion.

Where a borehole is formed using drilling fluid for maintaining the stability of a boring, the level of the water or fluid in the excavation shall be maintained so that the water or fluid pressure always exceeds the pressure exerted by the

soils and external ground water. The water or fluid level shall be maintained at a level not less than 2 metres above the level of ground water.

The reinforcement shall be placed as indicated on the Drawings. Reinforcement in the form of a cage shall be assembled with additional support, such as Spreader forks and lacings, necessary to form a rigid cage. Hoops, links or helical reinforcement shall fit closely around the main longitudinal bars and be bound to them by approved wire, the ends of which shall be turned into the interior of the pile or pour. Reinforcement shall be placed and maintained in position. The cover to all reinforcement for pile cap and bored cast in place pile shall be not less than 75 mm.

Joints in longitudinal steel bars shall be permitted unless otherwise specified. Joints in reinforcement shall be such that the full strength of the bar is effective across the joint and shall be made so that there is no relative displacement of the reinforcement during the construction of the pile.

Joints in longitudinal bars in piles with tension (for instance for test loading) shall be carried out by welding or other approved method.

Concrete to be placed under water or drilling fluid shall be placed by tremie equipment and shall not be discharged freely into the water or drilling fluid. The tremie equipment shall be designed to minimize the occurrence of entrapped air and other voids, so that it causes minimal surface disturbance, which is particularly important when a concrete-water interface exists. It shall be so designed that external projections are minimised, allowing the tremie to pass through reinforcing cages without causing damage. The internal face of the pipe of the tremie shall be free from projections. The tremie pipes shall meet the following requirements:

- (i) The tremie pipes shall be fabricated of heavy gage steel pipe to withstand all anticipated handling stress. Aluminium pipe shall not be used for placing concrete.
- (ii) Tremie pipes should have a diameter large enough to ensure that aggregates-caused blockage will not occur. The diameter of the tremie pipe shall be 200 mm to 300 mm.
- (iii) The tremie pipes shall be smooth internally.
- (iv) Since deep placement of concrete will be carried out, the tremie shall be made in sections/lengths with detachable joints that allow the upper sections/lengths to be removed as the placement progresses.
- (v) Sections may be joined by flanged, bolted connections (with gaskets) or may be screwed together. Whatever joint technique is selected, joints between tremie sections must be watertight. The joint system selected shall be tested for watertightness before beginning of concrete placement.
- (vi) The joint system to be used shall need approval of the Engineer.
- (vii) The tremie pipe should be marked to allow quick determination of the distance from the surface of the water to the mouth of the tremie.
- (viii) The tremie should be provided with adequately sized funnel or hopper to facilitate transfer of sufficient concrete from the delivery device to the tremie.

Before placing concrete, it shall be ensured that there is no accumulation of silt, other material, or heavily contaminated bentonite suspension at the base of the boring, which could impair the free flow of concrete from the pipe of the tremie. Flushing of boreholes before concreting with fresh drilling fluid/mud is preferred. A sample of the bentonite suspension shall be taken from the base of the boring using an approved sampling device. If the specific gravity of the suspension exceeds 1.25, the placing of concrete shall not proceed. In this event the Contractor shall modify the mud quality.

During and after concreting, care shall be taken to avoid damage to the concrete from pumping and dewatering operations.

The hopper and pipe of the tremie shall be clean and watertight throughout. The pipe shall extend to the base of the boring and a sliding plug or barrier shall be placed in the pipe to prevent direct contact between the first charge of concrete in the pipe of the tremie and the water or drilling fluid. The pipe shall at all times penetrate the concrete, which has previously been placed and shall not be withdrawn from the concrete-until completion of concreting. The bottom of the tremie pipe shall be embedded in the fresh concrete at least 2.0 metres and maintained at that depth throughout concreting. At all times a sufficient quantity of concrete shall be maintained within the pipe to ensure that the pressure from it exceeds that from the water or drilling fluid.

To ensure the quality of concrete being free from mud, clay lumps or any other undesirable materials mixed with concrete at the top portion of the pile, fresh concrete shall be overflowed sufficiently at the end of the each pour. The level of concrete poured at the end of concreting operation shall be at least 600 mm higher than the elevation of the pile at cut-off.

3.10.4.11.2 Concreting

In drilled shafts/cast-in-situ bored piles, concrete shall be placed only after excavation has been completed, inspected and accepted, and steel reinforcement accurately placed and adequately supported. Concrete shall be placed in one continuous operation in such a manner as to ensure the exclusion of any foreign matter and to secure a full sized shaft. Concrete shall not be placed through water except where tremie methods are approved. When depositing concrete from the top of pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centred at the top of the pile.

For large diameter holes concrete may be placed by tremie or by drop bottom bucket; for small diameter boreholes a tremie shall be utilized. In tremie concreting, toe of the tremie shall be set at a maximum of 150 mm above the bottom of the borehole. Maximum permissible siltation in bore hole prior to start of concrete operation shall be 75 mm. A slump of 125 mm to 150 mm shall be maintained for concreting by tremie. In case of tremie concreting for piles of smaller diameter and length up to 10 m, the minimum cement content shall be 350 kg/m³ of concrete. For larger diameter and/or deeper piles, the minimum cement content shall be 400 kg/m³ of concrete. See relevant sections of the code for further specification

For uncased concrete piles, if pile shafts are formed through unstable soil and concrete is placed in an open drill hole, a steel liner shall be inserted in the hole prior to placing concrete. If the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner to a sufficient height to offset any hydrostatic or lateral earth pressure.

If concrete is placed by pumping through a hollow stem auger, the auger shall not be permitted to rotate during withdrawal and shall be withdrawn in a steady continuous motion. Concrete pumping pressures shall be measured and shall be maintained high enough at all times to offset hydrostatic and lateral earth pressure. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. If the installation process of any pile is interrupted or a loss of concreting pressure occurs, the hole shall be redrilled to original depth and reformed.

Augured cast-in-situ pile shall not be installed within 6 pile diameters centre to centre of a pile filled with concrete less than 24 hours old. If concrete level in any completed pile drops, the pile shall be rejected and replaced. Bored cast-in-situ concrete piles shall not be drilled/bored within a clear distance of 3 m from an adjacent pile with concrete less than 48 hours old.

For under-reamed piles, the slump of concrete shall range between 100 mm and 150 mm for concreting in water free holes.

3.10.4.11.3 Concreting under Water

For concreting under water, the concrete shall contain at least 10 per cent more cement than that required for the same mix placed in the dry. The amount of coarse aggregate shall be not less than one and a half times, nor more

than two times, that of the fine aggregate. The materials shall be so proportioned as to produce a concrete having a slump of not less than 100 mm, nor more than 150 mm, except where plasticizing admixtures is used in which case, the slump may be 175 mm.

Successful placement of concrete under water requires preventing flow of water across or through the placement site. Once flow is controlled, the tremie placement consists of the following three basic steps:

- i) The first concrete placed is physically separated from the water by using a “rabbit” or go-devil in the pipe, or by having the pipe mouth capped or sealed and the pipe dewatered.
- ii) Once filled with concrete, the pipe is raised slightly to allow the “rabbit” to escape or to break the end seal. Concrete will then flow out and develop a mound around the mouth of the pipe. This is termed as “establishing a seal”.
- iii) Once the seal is established, fresh concrete is injected into the mass of existing concrete.

Two methods are normally used for the placement of concrete using tremie pipe, namely, the capped tremie pipe approach and the “rabbit” plug approach. In the capped tremie approach the tremie pipe should have a seal, consisting of a bottom plate or approved equal, that seals the bottom of the pipe until the pipe reaches the bottom of excavation. The tremie pipe should be filled with enough concrete before being raised off the bottom. The tremie pipe should then be raised a maximum of 150 mm (6 inch) to initiate flow. The tremie pipe should not be lifted further until a mound is established around the mouth of the tremie pipe. Initial lifting of the tremie should be done slowly to minimize disturbance of material surrounding the mouth of the tremie.

In the “rabbit” plug approach, open tremie pipe should be set on the bottom, the “rabbit” plug inserted at the top and then concrete should be added to the tremie slowly to force the “rabbit” downward separating the concrete from the water. Once the tremie pipe is fully charged and the “rabbit” reaches the mouth of the tremie, the tremie pipe should be lifted a maximum of 150 mm (6 inch) off the bottom to allow the “rabbit” to escape and to start the concrete flowing. After this, a tremie pipe should not be lifted again until a sufficient mound is established around the mouth of the tremie.

Tremies should be embedded in the fresh concrete a minimum of 1.0 to 1.5 m (3 to 5 ft) and maintained at that depth throughout concreting to prevent entry of water into the pipe. Rapid raising or lowering of the tremie pipe should not be allowed. All vertical movements of the tremie pipe must be done slowly and carefully to prevent “loss of seal”. If “loss of seal” occurs in a tremie, placement of concrete through the tremie must be halted immediately. The tremie pipe must be removed, the end plate must be restarted using the capped tremie approach. In order to prevent washing of concrete in place, a “rabbit” plug approach must not be used to restart a tremie after “loss of seal”.

Means of raising or lowering tremie pipes and of removing pipes smoothly without loss of concrete and without disturbing placed concrete or trapping air in the concrete shall be provided. Pipes shall not be moved horizontally while they are embedded in placed concrete or while they have concrete within them.

Underwater concrete shall be placed continuously for the whole of a pour to its full depth approved by the Engineer, without interruption by meal breaks, change of shift, movements of placing positions, and the like. Delays in placement may allow the concrete to stiffen and resist flow once placement resumes. The rate of pour from individual tremie shall be arranged so that concrete does not rise locally to a level greater than 500 mm above the average level of the surrounding concrete.

Tremie blockages which occur during placement should be cleared extremely carefully to prevent loss of seal. If a blockage occurs, the tremie should be quickly raised 150 to 600 mm (6 inch to 2 ft) and then lowered in an attempt to dislodge the blockage. The depth of pipe embedment must be closely monitored during all such attempts. If the blockage cannot be cleared readily, the tremie shall be removed, cleared, resealed, and restarted.

The volume of concrete in place should be monitored throughout the placement. Underruns are indicative of loss of tremie seal since the washed and segregated aggregates will occupy a greater volume. Overruns are indicative of loss of concrete from the inside of the steel pile.

3.11 FIELD TESTS FOR DRIVEN PILES AND DRILLED SHAFTS

3.11.1 Integrity Test

Low strain integrity testing of piles is a tool for quality control of long structural elements that function in a manner similar to foundation piles, regardless of their method of installation, provided that they are receptive to low strain impact testing. The test provides velocity (and optionally force) data, which assists evaluation of pile integrity and pile physical dimensions (i.e., cross-sectional area, length), continuity and consistency of pile material. The test does not give any information regarding the pile bearing capacity or about pile reinforcement. Integrity test principles have been well documented in literature (ASTM 5882-96; Klingmuller, 1993). There exist two methods of integrity testing, namely, Pulse Echo Method (PEM) and Transient Response Method (TRM). In Pulse Echo Method, the pile head motion is measured as a function of time. The time domain record is then evaluated for pile integrity. In Transient Response Method, the pile head motion and force (measured with an instrumented hammer) are measured as a function of time. The data are then evaluated usually in the frequency domain.

In order to check the structural integrity of the piles Integrity tests shall be performed on the piles in accordance with the procedure outlined in ASTM D5882. The test is carried out by pressing a transducer onto a pile top while striking the pile head with a hand hammer. The Sonic Integrity Testing (SIT)-system registers the impact of the hammer followed by the response of the pile and shows the display. If instructed by the operator, the signal will be stored in the memory of the SIT-system together with other information, such as pile number, date, time, site, amplification factor, filter length etc. The reflectograms are horizontally scaled and vertically amplified to compensate external soil friction, which facilitate the interpretation. Consequently, the reflection of the pile toe matches the length of the pile which will be confirmed by the SIT-system. In case of any defects, the exact location can be determined from the graph on the display.

For any project where pile has been installed, integrity tests shall be performed on 100% of the piles. Integrity testing may not identify all imperfections, but it can be used in identifying major defects within the effective length. In literature, there are many examples that highlight the success of low strain integrity testing (Klingmuller, 1993).

3.11.1.1 Some Factors Influencing Implication of Pile Integrity Test

(a) This sonic echo pile integrity testing or dynamic response method is based on measuring (or observing on an oscilloscope) the time it takes for a reflected compression stress wave to return to the top of the pile. Some waves will be reflected by a discontinuity in the pile shaft. When the compressive strength is known for the pile material involved, the depth to the discontinuity and the pile length can be determined.

(b) On the other hand, area of pile shaft and hence its diameter, is determined from impedance of wave response, while impedance in any section is a function of elastic modulus of pile material, shaft area and wave velocity propagating through that section. If the concrete material is uniform throughout the pile length, elastic modulus and the wave velocity (provided disturbance from other source of vibration nearby is insignificant) are constant for that pile. In that case, changes in impedance usually indicate changes of pile cross-sectional area.

(c) While evaluating pile integrity (i.e., pile length and shaft diameter), the wave velocity is usually assumed to be constant throughout pile length. Therefore, the reliability of integrity evaluation entirely depends on the pile material and its uniformity throughout shaft length while casting was done. Thus the length and diameter obtained from pile integrity test is merely an indication of the actual length and diameter of the tested piles.

(d) Besides, this test can only assess shaft integrity and gives no information for pile bearing capacity determination. However, if a large number of piles are tested, it is generally easy to focus the piles having unusual responses. Therefore, whenever an integrity testing is contemplated, consideration must be given to the limitations of the various methods/process of pile installation (i.e. pile driving or casting) and the possible need for further investigation (such as pile load test) to check the results of such testing.

It should be noted here that pile integrity test is an indicative test about the length and quality of concrete in the pile. This test does not give any idea about its actual load capacity. It is usually suggestive to substantiate the findings of integrity test by excavation or pull out of the pile to facilitate decisions about final acceptance or rejection of any pile. Because of the large cost involved in a pile load test, the necessity of integrity test in facilitating the selection of piles for load test is a rational approach for quality and safety assurance of piled foundations.

3.11.2 Axial Load Tests

Where accurate estimate of axial load carrying capacity of a pile is required tests in accordance with "Standard Test Method for Piles Under Static Compressive Load", (ASTM D1143) or equivalent shall be performed on individual piles. For a major project, at least 2% of piles (test piles plus service piles) shall be tested in each area of uniform subsoil conditions. Where necessary, additional piles may be load tested to establish the safe design capacity. The ultimate load carrying capacity of a single pile may be determined with reasonable accuracy from load testing. The load test on a pile shall not be carried out earlier than four weeks from the date of casting the pile. A minimum of one pile at each project shall be load tested for bored cast-in-situ piles.

Two principal types of test may be used for compression loading on piles - the constant rate of penetration (CRP) test and the maintained load (ML) test. The CRP test was developed by Whitaker (1953). The CRP method is essentially a test to determine the ultimate load on a pile and is therefore applied only to preliminary test piles or research type investigations where fundamental pile behaviour is being studied. In this test the compressive force is progressively increased to cause the pile to penetrate the soil at constant rate until failure occurs. The rate of penetration selected usually corresponds to that of shearing soil samples in unconfined compression tests. However, rate does not affect results significantly. In CRP test the recommended rates of penetration are 0.75 mm/min for friction piles in clay and 1.55 mm/min for piles end bearing in granular soil. The CRP test shall not be used for checking compliance with specification requirements for the maximum settlement at given stages of loading.

Maintained load (ML) test is so far the most usual one in practice. In the ML test the load is increased in stages to 1.5 times or twice the working load with time settlement curve recorded at each stage of loading and unloading. The general procedure is to apply static loads in increments of 25% of the anticipated design load. The ML test may also be taken to failure by progressively increasing the load in stages. In the ML test, the load test arrangements as specified in "Standard Test Method for Piles under Static Axial Compressive Load", (ASTM D1143), shall be followed. According to ASTM D1143 each load increment is maintained until the rate of settlement is not greater than 0.25 mm/hr or 2 hours is elapsed, whichever occurs first. After that the next load increment is applied. This procedure is followed for all increments of load. After the completion of loading if the test pile has not failed the total test load is removed any time after twelve hours if the butt settlement over one hour period is not greater than 0.25 mm otherwise the total test load is kept on the pile for 24 hours. After the required holding time, the test load is removed in decrement of 25% of the total test load with 1 hour between decrement. If failure occurs, jacking the pile is continued until the settlement equals 15% of the pile diameter or diagonal dimension. Details of the test procedure have been outlined in ASTM D1143.

Selection of an appropriate load test method shall be based on an evaluation of the anticipated types and duration of loads during service, and shall include consideration of the following:

- (a) The immediate goals of the load test (i.e., to proof load the foundation and verify design capacity)

- (b) The loads expected to act on the production foundation (compressive and/or uplift, dead and/or live), and the soil conditions predominant in the region of concern.
- (c) The local practice or traditional method

As a minimum, the written test procedures should include the following:

- (a) Apparatus for applying loads including reaction system and loading system.
- (b) Apparatus for measuring movements.
- (c) Apparatus for measuring loads.
- (d) Procedures for loading including rates of load application, load cycling and maximum load.
- (e) Procedures for measuring movements.
- (f) Safety requirements.
- (g) Data presentation requirements and methods of data analysis.
- (h) Drawings showing the procedures and materials to be used to construct the load test apparatus.

3.11.2.1 Load Test Evaluation Methods

A number of arbitrary or empirical methods are used to serve as criteria for determining the allowable and ultimate load carrying capacity from pile load test. Some are based on maximum permissible gross or net settlement as measured at the pile butt while the others are based on the performance of the pile during the progress of testing (Chellis, 1961; Whitaker, 1976; Poulos and Davis, 1980; Fuller, 1983). It is recommended to evaluate the load carrying capacity of piles and drilled shaft using any of the following methods:

- (a) Davission Offset Limit
- (b) British Standard Institution Criterion
- (c) Indian Standard Criteria
- (d) Butler-Hoy Criterion
- (e) Brinch-Hansen 90% Criterion
- (f) Other methods approved by engineer

The recommended criteria to be used for evaluating the ultimate and allowable load carrying capacity of piles and drilled shaft are summarized below.

- (a) A very useful method of computing the ultimate failure load has been reported by Davisson (1973). This method is based on offset method that defines the failure load. The elastic shortening of the pile, considered as point bearing, free standing column, is computed and plotted on the load-settlement curve, with the elastic shortening line passing through the origin. The slope of the elastic shortening line is 20° . An offset line is drawn parallel to the elastic line. The offset is usually 0.15 inch plus a quake factor, which is a function of pile tip diameter. For normal size piles, this factor is usually taken as 0.1D inch, where D is the diameter of pile in foot. The intersection of the offset line with the gross load-settlement curve determines the arbitrary ultimate failure load.
- (b) Terzaghi (1942) reported that the ultimate load capacity of a pile may be considered as that load which causes a settlement equal to 10% of the pile diameter. However, this criterion is limited to a case where no definite failure point or trend is indicated by the load-settlement curves (Singh, 1990). This criterion has been incorporated in BS 8004: 1986 of British Standard Institution (1986) which recommends that the ultimate load capacity of pile should be that which causes the pile to settle a depth of 10% of pile width or diameter.

- (c) According to the Code of Practice 2004 of British Standards Institution (1972), the allowable load capacity of pile should be 50% of the final load, which causes the pile to settle a depth of 10% of pile width or diameter.
- (d) According to IS: 2911 (Part-VI)-1979 ultimate load capacity of pile is smaller of the following two:
 - (i) Load corresponding to a settlement equal to 10% of the pile diameter in the case of normal uniform diameter pile or 7.5% of base diameter in case of under-reamed or large diameter cast in-situ pile.
 - (ii) Load corresponding to a settlement of 12 mm.
- (e) According to Indian Standard Code of practice (IS: 2911 – 1979), allowable load capacity of pile is smaller of the following:
 - (i) Two thirds of the final load at which the total settlement attains a value of 12 mm.
 - (ii) Half of the final load at which total settlement equal to 10% of the pile diameter in the case of normal uniform diameter pile or 7.5% of base diameter in case of under-reamed pile.
- (f) Butler and Hoy (1976) stated that the intersection of the tangent at the initial straight portion of the load-settlement curve and the tangent at a slope point of 1.27 mm/ton determines the arbitrary ultimate failure load.
- (g) The Brinch and Hansen (1963) proposed a definition for ultimate load capacity as that load for which the settlement is twice the settlement under 90 percent of the full test load.
- (h) Where failure occurs, the ultimate load may be taken to calculate the allowable load using a factor of safety of 2.0 to 2.5.

3.11.2.2 Some Factors Influencing Interpretations of Load Test Results

The following factors should be taken into account while interpreting the test results from pile load tests:

- (a) Potential residual loads (strains) in the pile which could influence the interpreted distribution of load along the pile shaft.
- (b) Possible interaction of friction loads from test pile with downward friction transferred to the soil from reaction piles obtaining part or all of their support in soil at levels above the tip level of the test pile.
- (c) Changes in pore water pressure in the soil caused by pile driving, construction fill and other construction operations which may influence the test results for frictional support in relatively impervious soils such as clay and silt.
- (d) Differences between conditions at time of testing and after final construction such as changes in grade groundwater level.
- (e) Potential loss of soil resistance from events such as excavation, or scour, or both of surrounding soil.
- (f) Possible difference in the performance of a pile in a group or of a pile group from that of a single isolated pile.
- (g) Affect on long term pile performance of factors such as creep, environmental effects on pile material, friction loads from swelling soils and strength losses.
- (h) Type of structure to be supported, including sensitivity of structure to movement and relations between live and dead loads.
- (i) Special testing procedures which may be required for the application of certain acceptance criteria or methods of interpretation.

- (j) Requirement that all conditions for non tested piles be basically identical to those for test pile including such thing as subsurface conditions, pile type, length, size and stiffness, and pile installation methods and equipment so that application or extrapolation of the test results to such other piles is valid

3.11.3 Uplift Capacity of Pile and Drilled Shaft

Where required by the design, the uplift capacity of pile and drilled shaft shall be determined by an approved method or analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D3689 (Standard Test Method for Individual Piles Under Static Axial Tensile Load). The maximum allowable uplift load shall not exceed the ultimate load capacity as determined using the results of load test conducted in accordance with ASTM D3689, divided by a factor of safety of 2.0. Where uplift is due to wind or seismic loading, the minimum factor of safety shall be 2.0 where capacity is determined by an analysis and 1.5 where capacity is determined by load tests.

For group pile subjected to uplift, the allowable working uplift load for the group shall be calculated by an approved method of analysis where the piles in the group are placed at centre-to-centre spacing of at least 2.5 times the least horizontal dimension of the largest pile, the allowable working uplift load for the group is permitted to be calculated as the lesser of the two:

- (a) The proposed individual working load times the number of piles in the group.
- (b) Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the embedded length of the pile.
- (c) One-half the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the embedded pile length plus one-half the total soil shear on the peripheral surface of the group

Uplift or tension test on piles subject to tension/uplift shall be performed by a continuous rate of uplift (CRU) or an incremental loading (i.e. ML) test. Where uplift loads are intermittent or cyclic in character, as in wave loading on a marine structure, it is recommended to adopt repetitive loading on the test pile. The tests shall be performed in accordance with "Standard Test Method for Individual Piles Under Static Axial Tensile Load", (ASTM D3689).

PART C: ADDITIONAL CONSIDERATIONS IN PLANNING, DESIGN AND CONSTRUCTION OF BUILDING FOUNDATIONS (SECTIONS 3.13 to 3.22)

3.12 EXCAVATION

Excavation for building foundation or for other purpose shall be done in a safe manner so that no danger to life and property prevails at any stage of the work or after completion. The requirements of this section shall be satisfied for all such works in addition to those of Sec 3.3 of Part 7.

Permanent excavations shall have retaining walls of sufficient strength made of steel, masonry, or reinforced concrete to retain the embankment, together with any surcharge load.

Excavations for any purpose shall not extend within 300 mm under any footing or foundation, unless such footing or foundation is first properly underpinned or protected against settlement.

3.12.1 Notice to Adjoining Property

Prior to any excavation close to an adjoining building in another property, a written notice shall be given to the owner of the adjoining property at least 10 days ahead of the date of excavation. The person undertaking the excavation shall, where necessary, incorporate adequate provisions and precautionary measures to ensure safety of the adjoining property and shall supply the details of such measures in the notice to the owner of the adjoining

property. He shall obtain approval of the Authority regarding the protective provisions, and permission of the owner of the adjoining property regarding the proposed excavation in writing.

The protective measures shall incorporate the following:

- (a) Where the level of the foundations of the adjoining structure is at or above the level of the bottom of the proposed excavation, the vertical load of the adjoining structure shall be supported by proper foundations, underpinning, or other equivalent means.
- (b) Where the level of the foundations of the adjoining structure is below the level of the bottom of the proposed excavation, provision shall be made to support any increased vertical or lateral load on the existing adjoining structure caused by the new construction.

If on giving the required notice, incorporating or proposing to incorporate the protective provisions which have duly been approved by the Authority, the owner of the adjoining property refuses to permit the proposed excavation or to allow necessary access and other facilities to the person undertaking the excavation for providing the necessary and approved protection to the adjoining property, the responsibility for any damage to the adjoining property due to excavation shall be that of the owner of the adjoining property.

3.12.2 Excavation Work

Every excavation shall be provided with safe means of entry and exit kept available at all times. When an excavation has been completed, or partly completed and discontinued, abandoned or interrupted, or the required permits have expired, the lot shall be filled and graded to eliminate all steep slopes, holes, obstructions or similar sources of hazard. Fill material shall consist of clean, noncombustible substances. The final surface shall be graded in such a manner as to drain the lot, eliminate pockets, prevent accumulation of water, and preclude any threat of damage to the foundations on the premises or on the adjoining property.

3.12.2.1 Methods of Protection

3.12.2.1.1 Shoring, Bracing and Sheet piling

With the exception of rock cuts, the sides of all excavations, including related or resulting embankments, 1.5 m or greater in depth or height measured from the level of the adjacent ground surface to the deepest point of excavation, shall be protected and maintained by shoring, bracing and sheeting, sheet piling, or other retaining structures. Alternatively, excavated slopes may be inclined not steeper than 1:1, or stepped so that the average slope is not steeper than forty five degrees with no step more than 1.5 m high, provided such slope does not endanger any structure, including subsurface structures. All sides or slopes of excavations or embankments shall be inspected after rainstorms, or any other hazard increasing event, and safe conditions shall be restored. Sheet piling and bracing needed in trench excavations shall have adequate strength to resist the possible forces resulting from earth or surcharge pressure. DESIGN OF PROTECTION SYSTEM SHALL BE CHECKED BY A GEOTECHNICAL ENGINEER.

3.12.2.1.2 Guard Rail

A guard rail or a solid enclosure at least 1 m high shall be provided along the open sides of excavations, except that such guard rail or solid enclosure may be omitted from a side or sides when access to the adjoining area is precluded, or where side slopes are one vertical to three horizontal or flatter.

3.12.2.2 Placing of Construction Material

Excavated materials and superimposed loads such as equipment, trucks, etc. shall not be placed closer to the edge of the excavation than a distance equal to one and one-half times the depth of such excavation, unless the excavation is in rock or the sides have been sloped or sheet piled (or sheeted) and shored to withstand the lateral force imposed by such superimposed load. When sheet piling is used, it shall extend at least 150 mm above the

natural level of the ground. In the case of open excavations with side slopes, the edge of excavation shall be taken as the toe of the slope.

3.12.2.3 Safety Regulations

Whenever subsurface operations are conducted that may impose loads or movement on adjoining property, such as driving of piles, compaction of soils, or soil densification, the effects of such operations on adjoining property and structures shall be considered. The owner of the property that may be affected shall be given 48 hours written notice of the intention to perform such operations. Where construction operations will cause changes in the ground water level under adjacent buildings, the effects of such changes on the stability and settlement of the adjacent foundation shall be investigated and provision made to prevent damage to such buildings. When a potential hazard exists, elevations of the adjacent buildings shall be recorded at intervals of twenty four hours or less to ascertain if movement has occurred. If so, necessary remedial action shall be undertaken immediately.

Whenever an excavation or fill is to be made that will affect safety, stability, or usability of adjoining properties or buildings, the adjoining properties or buildings shall be protected as required by the provisions of Sec 3.13.

On excavation, the soil material directly underlying footings, piers, and walls shall be inspected by an engineer/architect prior to construction of the footing. If such inspection indicates that the soil conditions do not conform to those assumed for the purposes of design and described on the plans, or are unsatisfactory due to disturbance, then additional excavation, reduction in allowable bearing pressure, or other remedial measures shall be adopted.

Except in cases where a proposed excavation will extend less than 1.5 m below grade, all underpinning operations and the construction and excavation of temporary or permanent cofferdams, caissons, braced excavation surfaces, or other constructions or excavations required for or affecting the support of adjacent properties or buildings shall be subject to controlled inspection. The details of underpinning, and construction of cofferdams, caissons, bracing or other constructions required for the support of adjacent properties or buildings shall be shown on the plans or prepared in the form of shop or detail drawings and shall be approved by the engineer who prepared the plans.

3.13 DEWATERING

All excavations shall be drained and the drainage maintained as long as the excavation continues or remains. Where necessary, pumping shall be used. No condition shall be created as a result of construction operations that will interfere with natural surface drainage. Water courses, drainage ditches, etc. shall not be obstructed by refuse, waste building materials, earth, stones, tree stumps, branches, or other debris that may interfere with surface drainage or cause the impoundment of surface water.

3.14 SLOPE STABILITY OF ADJOINING BUILDINGS

The possibility of overturning and sliding of the building shall be considered. The minimum factor of safety against overturning of the structure as a whole shall be 1.5. Stability against overturning shall be provided by the dead load of the building, the allowable uplift capacity of piling, anchors, weight of the soil directly overlying footings provided that such soil cannot be excavated without recourse to major modification of the building, or by any combination of these factors.

The minimum factor of safety against sliding of the structure under lateral load shall be 1.5. Resistance to lateral loads shall be provided by friction between the foundation and the underlying soil, passive earth pressure, batter piles or by plumb piles, subject to the following:

- (a) The resistance to lateral loads due to passive earth pressure shall not be taken into consideration where the abutting soil could be removed inadvertently by excavation.

- (b) In case of pile supported structures, frictional resistance between the foundation and the underlying soil shall be discounted.
- (c) The available resistance to friction between the foundation and the underlying soil shall be predicted on an assumed friction factor of 0.5. A greater value of the coefficient of friction may be used subject to verification by analysis and test.

The faces of cut and fill slopes shall be prepared and maintained to control erosion. The control may consist of effective planting. The protection for slopes shall be installed as soon as practicable. Where cut slopes are not subject to erosion due to erosion resistant character of the materials, such protection may be omitted.

Where necessary, check dams, cribbing, riprap or other devices or methods shall be employed to control erosion.

3.15 FILLS

3.15.1 Quality of Fill

The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris and large rocks. The backfill shall be placed in lifts and compacted in a manner which does not damage foundation, the waterproofing or damp-proofing material.

3.15.2 Placement of Fill

Fills to be used to support the foundation of any building or structure shall be placed in accordance with established engineering principle. Before placement of the fill, the existing ground surface shall be stripped off all organic growth, timber, rubbish and debris. After stripping, the ground surface shall be compacted. Materials for fill shall consist of sand, gravel, crushed stone, crushed earth, or a mixture of these. The fill material shall contain no particles exceeding 100 mm in the largest dimension. A soil investigation report and a report of satisfactory placement of fill, both acceptable to the Building Official shall be submitted. In an uncontrolled fill, the soil within the building area shall be explored using test pits. At least one test pit penetrating at least 2 m below the level of the bottom of the proposed foundation shall be provided for every 200 m² of building area. Wherever such test pits consistently indicate that the fill is composed of material that is free of voids and free of extensive inclusion of mud, organic materials such as paper, garbage, cans, metallic objects, or debris, the fill material shall be acceptable. Where the fill shows voids or inclusions as described above, either the fill shall be treated as having no presumptive bearing capacity, or the building shall incorporate adequate strength and stiffness to bridge such voids or inclusions or shall be articulated to prevent damage due to differential or localized settlement of the fill.

3.15.3 Specifications

Where foundations are to be placed on controlled fill materials, the fill must be compacted in layers not exceeding 300 mm. Clear specifications shall be provided for the range of water content, the degree of compaction to be achieved and the method of compaction that shall be followed. Such specifications shall be based on the shear strength requirement for the fill soil and allowable settlement estimate. The minimum density of controlled fill shall be 95% of the optimum density obtained from "Standard Test Method for Moisture-Density Relation of Soil and Soil-Aggregate Mixture using 10-lb (4.54 kg) Rammer and 18-in (457 mm) Drop", (ASTM D1557).

The degree of compaction achieved in a fill shall be obtained from in-situ density measurements. No new layer shall be placed unless a satisfactory density is attained in each layer.

3.16 PROTECTIVE Retaining Structures for Foundations/ Shore Piles

A retaining wall is a wall designed to resist lateral earth and/or fluid pressures, including any surcharge, in accordance with accepted engineering practice. Retaining walls for foundations shall be designed to ensure stability

against overturning, sliding, excessive foundation pressure and water uplift; and that they be designed for a safety factor of 1.5 against lateral sliding and overturning. Generally sheet pile retaining walls are used for construction raft foundations for buildings. Taller sheet piles may need a tie back anchor driven and anchored behind the soil of the sheet pile retaining wall.

3.17 WATERPROOFING AND DAMP-PROOFING

Walls or portions thereof that retain earth and enclose interior spaces, and floors below grade shall be waterproofed and damp-proofed, with the exception of those spaces where such omission is not detrimental to the building or occupancy. The roof is also required to be waterproofed. The owner shall perform a subsurface investigation to determine the possibility of the ground water table rising above the proposed elevation of the floor or floors below grade unless satisfactory data from adjacent areas demonstrate that ground water has not been a problem.

There may arise two situations: (i) where no hydrostatic pressure occurs and (ii) where hydrostatic pressure occurs. Where hydrostatic pressure conditions exist, floors and walls below finished ground level shall be waterproofed in accordance with Sec 3.13.1 below. Where hydrostatic pressure conditions do not exist, damp-proofing and perimeter drainage shall be provided in accordance with Sec 3.13.2 below. In addition, the damp-proofing and waterproofing shall also meet the requirements of Sec 3.13.3. All damp-proofing and waterproofing materials shall conform to the requirements of Sec 2.16.7 of Part 5.

3.17.1.1 Waterproofing where Hydrostatic Pressure Occurs

Where ground water investigation indicates that a hydrostatic pressure condition exists, or is likely to occur, walls and floors shall be waterproofed in accordance with this section.

3.17.1.2 Floor Waterproofing

Floors required to be waterproofed shall be of concrete and shall be designed and constructed to withstand the anticipated hydrostatic pressure.

Waterproofing of the floor shall be accomplished by placing under the slab a membrane of rubberized asphalt, or butyl rubber, or polymer modified asphalt, or neoprene, or not less than 0.15 mm polyvinyl chloride or polyethylene, or other approved materials, capable of bridging nonstructural cracks. Joints in the membrane shall be lapped not less than 150 mm and sealed in an approved manner.

3.17.1.3 Wall Waterproofing

Walls required to be waterproofed shall be of concrete or masonry designed to withstand the anticipated hydrostatic pressure and other lateral loads. Prior to the application of waterproofing materials on concrete walls, all holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be pargeted on the exterior surface below ground level with not less than 10 mm of Portland cement mortar. The pargeting shall be continued to the foundation. Pargeting of unit masonry walls is not required where a material is approved for direct application to the masonry.

Waterproofing shall be applied from a point 300 mm above the maximum elevation of the ground water table down to the top of the spread portion of the foundation. The remainder of the wall up to a level not less than 150 mm above finished grade shall be damp-proofed.

Wall waterproofing materials shall consist of two-ply hot-mopped felts, not less than 0.15 mm polyvinyl chloride, 1.0 mm polymer modified asphalt, 0.15 mm polyethylene or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane shall be lapped not less than 150 mm and sealed in an approved manner.

Joints in walls and floors, joints between the wall and the floor, and penetrations of the wall and floor shall be made watertight utilizing established methods and materials.

3.17.1.4 Damp-proofing with no Hydrostatic Pressure

Where hydrostatic pressure will not occur, floors and walls shall be damp-proofed and a subsoil drainage system shall be installed as described below:

3.17.1.5 Floor Damp-proofing

For floors, damp-proofing materials shall be installed between the floor and base materials. The base material shall not be less than 100 mm in thickness consisting of gravel or crushed stone containing not more than 10 per cent material that passes a 4.76 mm sieve. Where a site is located in well drained gravel or sand/gravel mixture, a floor base is not required. When the finished ground level is below the floor level for more than 25 per cent of the perimeter of the building, the base material need not be provided. Where a separate floor is provided above a concrete slab the damp-proofing may be installed on top of the slab.

Damp-proofing materials, where installed beneath the slab, shall consist of not less than 0.15 mm polyethylene with joints lapped not less than 150 mm, or other approved methods or materials. Where permitted to be installed on top of the slab, damp-proofing shall consist of mopped on bitumen, not less than 0.1 mm polyethylene, or other approved methods or materials. Joints in membranes shall be lapped not less than 150 mm and sealed in an approved manner.

3.17.1.6 Wall Damp-proofing

For walls, damp-proofing materials shall be installed and shall extend from a point 150 mm above grade, down to the top of the spread portion of the foundation.

Wall damp-proofing material shall consist of a bituminous material, acrylic modified cement base coating, rubberized asphalt, polymer-modified asphalt, butyl rubber, or other approved materials capable of bridging nonstructural cracks.

3.17.1.7 Perimeter Drain

A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10 per cent material that passes through a 4.76 mm sieve. The drain shall extend a minimum of 300 mm beyond the outside edge of the foundation. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 150 mm above the top of the foundation. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 50 mm of gravel or crushed stone complying with this section, and shall be covered with not less than 150 mm of the same material.

The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system. Where a site is located in well drained gravel or sand/gravel mixture, a dedicated drainage system is not required. When the finished ground level is below the floor level for more than 25 per cent of the perimeter of the building, the foundation drain need be provided only around that portion of the building where the ground level is above the floor level.

3.17.2 Other Damp-proofing and Waterproofing Requirements

3.17.2.1 Placement of Backfill

The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris and large rocks. The backfill shall be placed in lifts and compacted in a manner which does not damage the waterproofing or damp-proofing material or structurally damage the wall.

3.17.2.2 Site Grading

The ground immediately adjacent to the foundation shall be sloped away from the building at a slope not less than 1 unit vertical in 12 units horizontal (1:12) for a minimum distance of 2.5 m measured perpendicular to the face of the wall or an alternative method of diverting water away from the foundation shall be used. Consideration shall be given to possible additional settlement of the backfill when establishing the final ground level adjacent to the foundation.

3.17.2.3 Erosion Protection

Where water impacts the ground from the edge of the roof, down spout, scupper, valley or other rainwater collection or diversion device, provisions shall be used to prevent soil erosion and direct the water away from the foundation.

3.18 FOUNDATION ON SLOPES

3.18.1 Footings on Slopes

Where footings are to be founded on a slope, the distance of the sloping surface at the base level of the footing measured from the centre of the footing shall not be less than twice the width of the footing.

When adjacent footings are to be placed at different levels, the distance between the edges of footings shall be such as to prevent undesirable overlapping of structures in soil and disturbance of the soil under the higher footing due to excavation of the lower footing.

On a sloping site, footing shall be on a horizontal bearing and stepped. At all changes of levels, footings shall be lapped for a distance of at least equal to the thickness of foundation or three times the height of step, whichever is greater. Adequate precautions shall be taken to prevent tendency for the upper layers of soil to move downhill.

3.19 FOUNDATIONS ON FILLS AND PROBLEMATIC SOILS

3.19.1 Footings on Filled up Ground

Footings shall not be constructed on loosely filled up ground with non uniform density or consistency, unless adequate strengthening of the soil is made by applying ground improvement techniques.

3.19.2 Ground Improvement

In poor and weak subsoils, the design of shallow foundation for structures and equipment may present problems with respect to both sizing of foundation as well as control of foundation settlements. A viable alternative in certain situations developed over recent years is to improve the subsoil to an extent that the subsoil would develop an adequate bearing capacity and foundations constructed after subsoil improvement would have resultant settlements within acceptable limits. Selection of ground improvement techniques may be done in accordance with good practice.

3.19.3 Soil Reinforcement

Use of suitable geo-synthetics/geo-textiles may be made in an approved manner for ground improvement where applicable based on good practice.

3.20 FOUNDATION DESIGN FOR DYNAMIC FORCES

3.20.1 Effect of Dynamic Forces

Where machinery operations or other vibrations are transmitted through foundation, consideration shall be given in the foundation design to prevent detrimental disturbance of the soil.

Impact forces shall be neglected in foundation design except for foundations bearing on loose granular soils, foundations supporting cranes, heavy machinery and moving equipment, or where the ratio of live load causing the impact to the dead load exceeds 50%.

3.20.2 Machine Foundation

Machine foundations are subjected to the dynamic forces caused by the machine. These dynamic forces are transmitted to the foundation supporting the machine. Although the moving parts of the machine are generally balanced, there is always some unbalance in practice which causes an eccentricity of rotating parts. This produces an oscillating force. The machine foundation must satisfy the criteria for dynamic loading in addition to that for static loading.

3.20.2.1 Types of Machine Foundations

Basically, there are three types of machine foundation:

- (a) Machines which produce a periodic unbalanced force, such as reciprocating engines and compressors. The speed of such machines is generally less than 600 rpm. In these machines, the rotary motion of the crank is converted into the translatory motion. The unbalanced force varies sinusoidal.
- (b) Machines which produce impact loads, such as forge hammers and punch presses. In these machines, the dynamic force attains a peak value in a very short time and then dies out gradually. The response is a pulsating curve. It vanishes before the next pulse. The speed is usually between 60 to 150 blows per minute.
- (c) High speed machines, such as turbines, and rotary compressors. The speed of such machines is very high; sometimes, it is even more than 3000 rpm.

The following four types of machine foundations are commonly used.

- (a) Block Type: This type of machine foundation consists of a pedestal resting on a footing (Fig. 6.3.3a). The foundation has a large mass and a small natural frequency.
- (b) Box Type: The foundation consists of a hollow concrete block (Fig. 6.3.3b). The mass of the foundation is less than that in the block type and the natural frequency is increased.
- (c) Wall Type: A wall type of foundation consists of a pair of walls having a top slab. The machine rests on the top slab (Fig. 6.3.3c).
- (d) Framed Type: This type of foundation consists of vertical columns having a horizontal frame at their tops. The machine is supported on the frame (Fig. 6.3.3d).

Machines which produce periodical and impulsive forces at low speeds are generally provided with a block type foundation. Framed type foundations are generally used for the machines working at high speeds and for those of

the rotating types. Some machines which induce very little dynamic forces, such as lathes, need not be provided with a machine foundation. Such machines may be directly bolted to the floor.

3.20.2.2 Design Considerations

For satisfactory performance, machine foundations should satisfy the following requirements: (i) resonance is avoided, (ii) bearing capacity and settlement are safe, and (iii) there is an adequate vibration and shock isolation. Avoidance of resonance is discussed in this section.

(I) Resonance: Based on their operating frequencies, the machines are classified as (i) low speed having frequency less than 300 revolutions per minute (rpm), (ii) medium speed, frequency 300 to 1000 rpm, and (iii) high speed, frequency greater than 1000 rpm. To avoid resonance, the natural frequency (or the resonant frequency) of the machine foundation-soil system must be either very large or very small compared to the operating speed of the machine.

- (a) **Low speed machines ($f_1 < 300$ rpm):** Provide a foundation with a natural frequency at least twice the operating frequency, i.e., the frequency ratio $r (=f_d/f_n)$ is less than 0.5. Natural frequency can be increased (i) by increasing base area or reducing total static weight of the foundation, (ii) by increasing modulus of shear rigidity of the soil by compaction, grouting or injection, (iii) by using piles to provide the required foundation stiffness.
- (b) **High speed machines ($f_1 > 1000$ rpm):** Provide a foundation with natural frequency not higher than one-half of the operating value, i.e., frequency ratio $r \geq 2$. Natural frequency can be decreased by increasing weight of foundation. During starting and stopping, the machine will operate briefly at resonant frequency (f_r) of the foundation. Probable amplitude is computed at both f_r and f_1 and compared with allowable values to determine if the foundation arrangement must be altered.

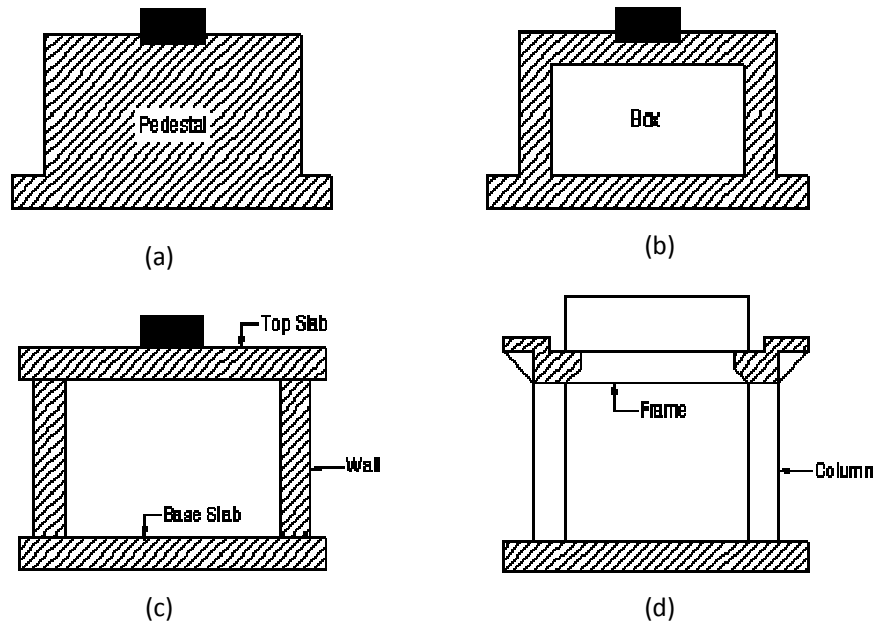


Fig. 6.3.3. Types of machine foundations (a) Block Type (b) Box Type (c) Wall Type (d) Framed Type.

(2) Types of foundations. Considering their structural forms, the machine foundations, in general, are of the following types: (i) box foundation consisting of a pedestal of concrete, (ii) box foundation consisting of a hollow concrete block, (iii) wall foundation consisting of a pair of walls supporting the machine. (iv) framed foundation consisting of vertical columns and a top horizontal frame work which forms the seat of essential machinery.

Low speed machines (e.g., forge hammers, presses, low speed reciprocating engines and compressors) are generally supported on block foundation having a large contact area with soil. Medium speed machines (e.g., reciprocating diesel and gas engines) also have, in general, block foundations resting on springs or suitable elastic pads. High speed and rotating type of machines (e.g., internal combustion engines, electric motors, and turbo generator machines) are generally mounted on framed foundations. Other high speed machines are placed on block foundations. As far as possible, the C.G. of the whole system and the centroid of the base area should be on the same vertical axis. At the most an eccentricity of 5% could be allowed.

(3) Permissible amplitude. Many times the permissible amplitude at operating speed is specified by the manufactures. If not specified, the following values may be adopted for guidance (i) low speed machines. ($f_1 < 500$ rpm), horizontal and vertical vibrations, $A = 0.25$ mm; (ii) operating speed $f_1 = 500 - 1500$ rpm, $A = 0.4$ mm to 0.6 mm for horizontal, and $A = 0.7$ mm to 0.9 mm for vertical mode of vibration; (iii) operating speed f_1 up to 3000 rpm, $A = 0.2$ mm for horizontal and $A = 0.5$ mm for vertical vibrations (iv) hammer foundations, $A = 10$ mm.

3.20.2.3 Design Methods

The various design methods can be grouped as follows: (i) empirical and semi-empirical methods, (ii) methods considering soil as a spring and (iii) methods considering soil as a semi-infinite elastic mass (elastic half-space-approach) and its equivalent lumped parameter method. The lumped parameter method is currently preferred and will be described here.

A good machine foundation should satisfy the following criteria.

- (a) Like ordinary foundations, it should be safe against shear failure caused by superimposed loads, and also the settlements should be within the safe limits.
- (b) The soil pressure should normally not exceed 80% of the allowable pressure for static loading.
- (c) There should be no possibility of resonance. The natural frequency of the foundation should be either greater than or smaller than the operating frequency of the machine.
- (d) The amplitudes under service condition should be within the permissible limits for the machine.
- (e) The combined centre of gravity of the machine and the foundation should be on the vertical line passing through the centre of gravity of the base plane.
- (f) Machine foundation should be taken to a level lower than the level of the foundation of the, adjacent buildings and should be properly separated.
- (g) The vibrations induced should neither be annoying to the persons nor detrimental to other structures.
- (h) Richart (1967) developed a plot for vertical vibrations, which is generally taken as a guide for various limits of frequency and amplitude which has been presented in Fig. 6.3.4.
- (i) The depth of the ground-water table should be at least one-fourth of the width of the foundation below the base place.

3.20.2.4 Vibration Analysis of a Machine Foundation:

Although a machine foundation has 6 degree of freedom, it is assumed to have a single degree of freedom for a simplified analysis. Fig 6.3.5 shows a machine foundation supported on a soil mass. In this case, the mass m_f lumps together the mass of the machine and the mass of foundation. The total mass m_f acts at the centre of gravity of the system. The mass is under the supporting action of the soil. The elastic action can be lumped together into a single elastic spring with a stiffness k . Likewise; all the resistance to motion is lumped into the damping coefficient c . Thus the machine foundation reduces to a single mass having one degree of freedom. The analysis of damped, forced vibration is, therefore, applicable to the machine foundation.

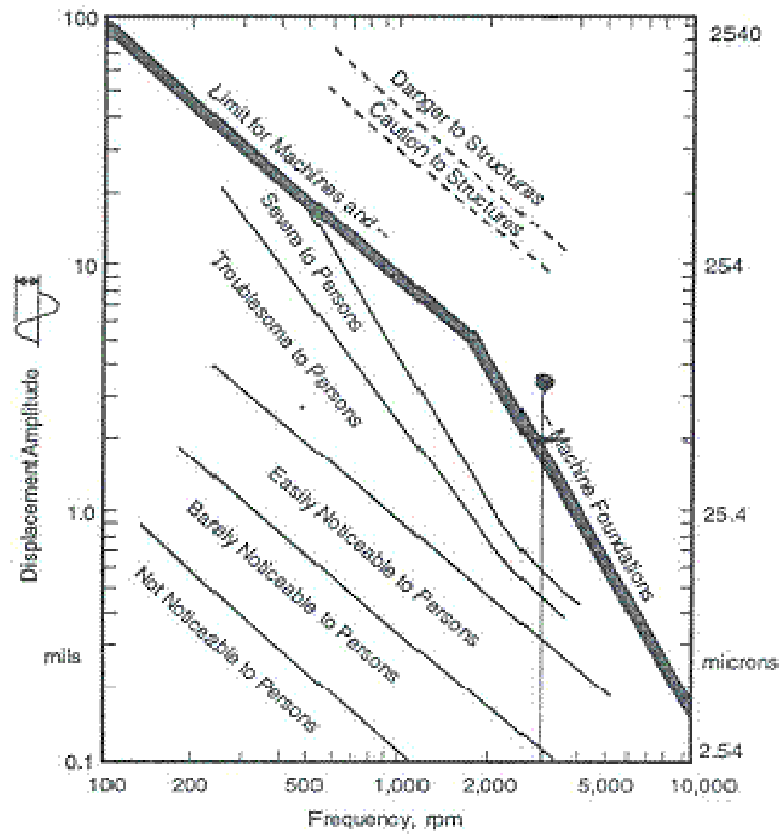


Fig. 6.3.4. Limits of frequency and amplitudes (Richart's chart)

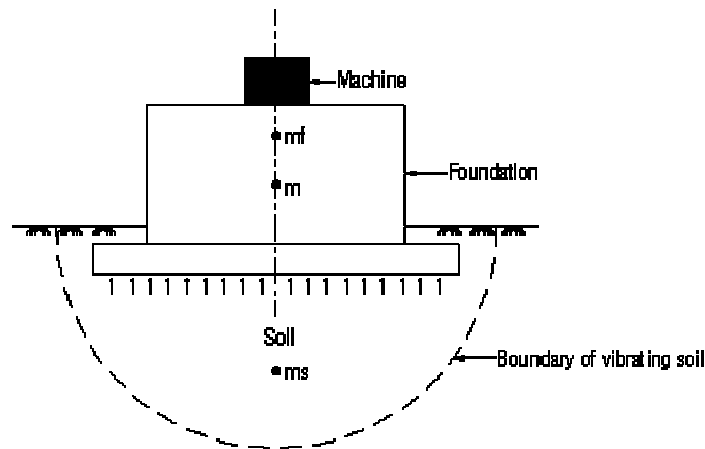


Fig. 6.3.5. Machine foundation supported on a soil mass

3.20.2.4.1 Determination of Parameters

For vibration analysis of a machine foundation, the parameters m , c and k are required. These parameters can be determined as under.

- (a) **Mass (m):** When a machine vibrates, some portion of the supporting soil mass also vibrates. The vibrating soil is known as the participating mass or in-phase soil mass. Therefore, the total mass of the system is equal to the mass of the foundation block and machine (m_f) and the mass (m_s) of the participating soil. Thus

$$m = m_f + m_s \quad (6.3.51)$$

Unfortunately, there is no rational method to determine the magnitude of m_s . It is usually related to the mass of the soil in the pressure bulb. The value of m_s generally varies between zero and m_f . In other words, the total mass (m) varies between m_f and $2m_f$ in most cases.

(b) Spring stiffness (k): The spring stiffness depends upon the type of soil, embedment of the foundation block, the contact area and the contact pressure distribution. The following are the common methods.

- i) **Laboratory test:** A triaxial test with vertical vibrations is conducted to determine Young's modulus E . Alternatively, the modulus of rigidity (G) is determined conducting the test under torsional vibration, and E is obtained indirectly from the relation, $E = 2G(1 + \mu)$, where μ is Poisson's ratio. The stiffness (k) is determined as

$$k = \frac{AE}{L} \quad (6.3.52)$$

where, A = cross-sectional area of the specimen, and L = length of the specimen.

- ii) **Barkan's method:** The stiffness can also be obtained from the value of E using the following relation given by Barken.

$$k = \frac{1.13E}{1 - \mu} \sqrt{A} \quad (6.3.53)$$

Where, A = base area of the machine, i.e. area of contact.

- iii) **Plate load test:** A repeated plate load test is conducted and the stiffness of the soil (k_p) is found as the slope of the load-deformation curve. The spring constant k of the foundation is as under.

For cohesive soils: $k = k_p \left(\frac{B}{B_p} \right)$ (6.3.54)

For cohesionless soil: $k = k_p \left(\frac{B + 0.3}{B_p + 0.3} \right)^2$ (6.3.55)

Where, B is the width of foundation. Alternatively, spring constant can be obtained from the subgrade modulus (k_s), as

$$k = K_s A \quad (6.3.56)$$

Where, A = area of foundation.

- iv) **Resonance test:** The resonance frequency (f_n) is obtained using a vibrator of mass m set up on a steel plate supported on the ground. The spring stiffness obtained from the relation

$$f_n = \frac{\omega_n}{2\pi} = \frac{1}{2\pi} \sqrt{k/m} = 4\pi^2 f_n m \quad (6.3.57)$$

II. **Damping constant (c):** Damping is due to dissipation of vibration energy, which occurs mainly because of the following reasons.

- i) Internal friction loss due to hysteresis and viscous effects.
ii) Radiational loss due to propagation of waves through soil.

The damping factor D for an under-damped system can be determined in the laboratory. Vibration response is plotted and the logarithmic decrement δ is found from the plot, as

$$\delta = \frac{2\pi D}{\sqrt{1-D^2}} \Rightarrow D = \frac{\delta}{2\pi} \quad (6.3.58)$$

The damping factor D may also be obtained from the area of the hysteresis loop of the load displacement curve, as

$$D = \frac{\Delta W}{W} \quad (6.3.59)$$

Where, W = total work done; and ΔW = work lost hysteresis. The value of D for most soils generally varies between 0.01 and 0.1.

3.21 GEO-HAZARD ANALYSIS FOR BUILDINGS

Geo-hazard analysis of buildings include design considerations for possible landslides, ground subsidence, earthquakes and other seismic events, erosion and scour, construction in toxic and/or contaminated landfills, groundwater contamination etc. A preliminary review of the selected site should be carried out for existence of any of the above mentioned geo-hazard in the area. A detailed analysis may be carried out only if the preliminary review indicates a significant threat for the building which may exist from any of the above mentioned potential geo-hazard at the selected location for the building. See relevant section for details.

Chapter 4

BAMBOO

4.1 SCOPE

This Section relates to the use of bamboo in construction as structural elements, nonstructural elements and also for temporary works in structures or elements of the structure, ensuring quality and effectiveness of design and construction using bamboo. It covers minimum strength data, dimensional and grading requirements, seasoning, preservative treatment, design and jointing techniques with bamboo which would facilitate scientific application and long-term performance of structures. It also covers guidelines so as to ensure proper procurement, storage, precautions and design limitations on bamboo.

4.2 TERMINOLOGY

For the purpose of this Section, the following definitions shall apply.

4.2.1 Anatomical Purpose Definitions

Bamboo — Tall perennial grasses found in tropical and sub-tropical regions. They belong to the family Poaceae and sub-family Bambusoideae.

Bamboo Culm — A single shoot of bamboo usually hollow except at nodes which are often swollen.

Bamboo Clump — A cluster of bamboo culms emanating from two or more rhizomes in the same place.

Cellulose — A carbohydrate, forming the fundamental material of all plants and a main source of the mechanical properties of biological materials.

Cell — A fundamental structural unit of plant and animal life, consisting of cytoplasm and usually enclosing a central nucleus and being surrounded by a membrane (animal) or a rigid cell wall (plant).

Cross Wall — A wall at the node closing the whole inside circumference and completely separating the hollow cavity below from that above.

Hemi Cellulose — The polysaccharides consisting of only 150 to 200 sugar molecules, also much less than the 10000 of cellulose.

Lignin — A polymer of phenyl propane units, in its simple form (C₆H₅CH₂CH₂CH₃).

Sliver — Thin strips of bamboo processed from bamboo culm.

Tissue — Group of cells, which in higher plants consist of (a) Parenchyma — a soft cell of higher plants as found in stem pith or fruit pulp, (b) Epidermis — the outermost layer of cells covering the surface of a plant, when there are several layers of tissue.

4.2.2 Structural Purpose Definitions

Bamboo Mat Board — A board made of two or more bamboo mats bonded with an adhesive.

Beam — A structural member which supports load primarily by its internal resistance to bending.

Breaking Strength — A term loosely applied to a given structural member with respect to the ultimate load it can sustain under a given set of conditions.

Bundle-Column — A column consisting of three or more number of culms bound as integrated unit with wire or strap type of fastenings.

Centre Internode — A test specimen having its centre between two nodes.

Characteristic Load — The value of loads which has a 95 percent probability of not exceeding during the life of the structure.

Characteristic Strength — The strength of the material below which not more than 5 percent of the test results are expected to fall.

Cleavability — The ease with which bamboo can be split along the longitudinal axis. The action of splitting is known as cleavage.

Column — A structural member which supports axial load primarily by inducing compressive stress along the fibres.

Common Rafter — A roof member which supports roof battens and roof coverings, such as boarding and sheeting.

Curvature — The deviation from the straightness of the culm.

Delamination — Separation of mats through failure of glue.

End Distance — The distance measured parallel to the fibres of the bamboo from the centre of the fastener to the closest end of the member.

Flatten Bamboo — Bamboo consisting of culms that have been cut and unfolded till it is flat. The culm thus is finally spread open, the diaphragms (cross walls) at nodes removed and pressed flat.

Full Culm — The naturally available circular section/shape.

Fundamental or Ultimate Stress — The stress which is determined on a specified type/size of culms of bamboo, in accordance with standard practice and does not take into account the effects of naturally occurring characteristics and other factors.

Inner Diameter — Diameter of internal cavity of a hollow piece of bamboo.

Inside Location — Position in buildings in which bamboo remains continuously dry or protected from weather.

Joint — A connection between two or more bamboo structural elements.

Joist — A beam directly supporting floor, ceiling or roof of a structure.

Length of Internode — Distance between adjacent nodes.

Loaded End or Compression End Distance — The distance measured from the centre of the fastener to the end towards which the load induced by the fastener acts.

Matchet — A light cutting and slashing tool in the form of a large knife.

Mat — A woven sheet made using thin slivers.

Mortise and Tenon — A joint in which the reduced end (tenon) of one member fits into the corresponding slot (mortise) of the other.

Net Section — Section obtained by deducting from the gross cross-section (A), the projected areas of all materials removed by boring, grooving or other means.

Node — The place in a bamboo culm where branches sprout and a diaphragm is inside the culm and the walls on both sides of node are thicker.

Outer Diameter — Diameter of a cross-section of a piece of bamboo measured from two opposite points on the outer surface.

Outside Location — Position in building in which bamboos are occasionally subjected to wetting and drying as in case of open sheds and outdoor exposed structures,

Permissible Stress — Stress obtained after applying factor of safety to the ultimate or basic stress.

Principal Rafter — A roof member which supports purlins.

Purlins — A roof member directly supporting roof covering or common rafter and roof battens.

Roof Battens — A roof member directly supporting tiles, corrugated sheets, slates or other roofing materials.

Roof Skeleton — The skeleton consisting of bamboo truss or rafter over which solid bamboo purlins are laid and lashed to the rafter or top chord of a truss by means of galvanized iron wire, cane, grass, bamboo leaves, etc.

Slenderness Ratio — The ratio of the length of member to the radius of gyration is known as slenderness ratio of member. (The length of the member is the equivalent length due to end conditions).

Splits — The pieces made from quarters by dividing the quarters radially and cutting longitudinally.

Taper — The ratio of difference between minimum and maximum outer diameter to length.

Unloaded End Distance — The end distance opposite to the loaded end

Wall Thickness — Half the difference between outer diameter and inner diameter of the piece at any cross-section.

Wet Location — Position in buildings in which the bamboos are almost continuously damp, wet or in contact with earth or water, such as piles and bamboo foundations.

4.2.3 Definitions Relating to Defects

Bamboo Bore/GHOON Hole — The defect caused by bamboo GHOON beetle (*Dinoderus* spp. Bostychdae), which attacks felled culms.

Crookedness — A localized deviation from the straightness in a piece of bamboo.

Discoloration — A change from the normal colour of the bamboo which does not impair the strength of bamboo or bamboo composite products.

4.2.4 Definitions Relating to Drying Degradates

Collapse — The defect occurring on account of excessive shrinkage, particularly in thick walled immature bamboo. When the bamboo wall shrinks, the outer layers containing a larger concentration of strong fibro-vascular bundles set the weaker interior portion embedded in parenchyma in tension, causing the latter to develop cracks. The interior crack develops into a wide split resulting in a depression on the outer surface. This defect also reduces the structural strength of round bamboo.

End Splitting — A split at the end of a bamboo. This is not so common a defect as drying occurs both from outer and interior wall surfaces of bamboo as well as the end at the open ends.

Surface Cracking — Fine surface cracks not detrimental to strength, However, the cracking which occurs at the nodes reduces the structural strength.

Wrinkled and Deformed Surface — Deformation in cross-section, during drying, which occurs in immature round bamboos of most species; in thick walled pieces, besides this deformation the outer surface becomes uneven and wrinkled. Very often the interior wall develops a crack below these wrinkles, running parallel to the axis.

4.3 SYMBOLS

For the purpose of this Section, the following letter symbols shall have the meaning indicated against each, unless otherwise stated:

A = Cross-sectional area of bamboo (perpendicular to the direction of the principal fibres and vessels), mm²

$$A = \frac{\pi}{4}(D^2 - d^2)$$

D = Outer diameter, mm

d = Inner diameter, mm

E = Modulus of elasticity in bending, N/mm²

f_c = Calculated stress in axial compression, N/mm²

f_{cp} = Permissible stress in compression along the fibres, N/mm²

$$I = \text{Moment of inertia, mm}^4 = \frac{\pi}{64}(D^2 - d^2)$$

l = Unsupported length of column

M = Moisture content, percent

$$r = \text{Radius of gyration} = \sqrt{I/A}$$

R' = Modulus of rupture, N/mm²

W = Wall thickness, mm

Z = Section modulus, mm³

δ = Deflection or deformation, mm.

4.4 MATERIALS

4.4.1 Species of Bamboo

In Bangladesh, four species are widely used, hence studied for the mechanical properties as tabulated in Table 6.4.1-6.4.4 for top, bottom and middle positions. Table 6.4.5 further summarize the average mechanical properties of 21 bamboo species.

Table 6.4.1: Moisture content and specific gravity values of four bamboos species at different height positions (average of five bamboo specimens)

Species	Moisture content (%)			Specific Gravity (based on oven dry weight and at different volumes)					
	bottom	middle	top	Green volumes			Oven dry volumes		
				bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	129	118	104	0.48	0.49	0.51	0.66	0.69	0.74
Mitinga (Bambusa tulda)	108	92	86	0.54	0.58	0.61	0.75	0.79	0.83
Bethua (Bambusa polymorpha)	104	93	79	0.55	0.57	0.61	0.79	0.81	0.54
Borak (Bambusa balcooa)	100	84	66	0.57	0.64	0.74	0.79	0.84	0.85

Table 6.4.2: Shrinkages of wall thickness and in diameter of four bamboo species at different height positions

Species	Shrinkage in wall thickness (%)						Shrinkage in diameter (%)		
	From green to 12% mc			From green to oven dry condition			From green to 12% mc		
	bottom	middle	top	bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	9.6	8.1	5.9	13.2	10.7	8.7	4.8	3.0	2.4
Mitinga (Bambusa tulda)	11.9	7.3	4.9	14.9	9.6	7.6	3.9	3.5	2.6
Bethua (Bambusa polymorpha)	10.7	6.5	5.1	12.1	10.1	8.2	7.3	5.5	4.1
Borak (Bambusa balcooa)	11.1	7.6	4.8	13.7	11.1	8.4	4.2	3.4	2.5

Table 6.4.3: Compressive strength of four bamboo species at different height positions

Species	Compression parallel to the grain (kg/cm ²)					
	green			airdry		
	bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	257	287	301	346	387	417
Mitinga (Bambusa tulda)	403	466	513	529	596	620
Bethua (Bambusa polymorpha)	320	361	419	452	512	534

Borak (Bambusa balcooa)	394	459	506	510	536	573

Table 6.4.4: Modulus of elasticity and modulus of rupture values of four bamboo species at different height positions

Species	Modulus of elasticity (1000 kg/cm ²)						Modulus of rupture (kg/cm ²)					
	green			airdry			green			airdry		
	bottom	middle	top	bottom	middle	top	bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	119	131	169	131	150	224	541	459	415	721	580	530
Mitinga (Bambusa tulda)	105	138	147	114	140	168	710	595	542	883	745	671
Bethua (Bambusa polymorpha)	61	65	82	60	70	96	469	426	373	566	468	414
Borak (Bambusa balcooa)	72	92	103	93	108	127	850	712	624	926	787	696

4.4.2 Grouping

Sixteen species of bamboo are suitable for structural applications and classified into three groups, namely, Group A, Group B and Group C as given in Table 6.4.6.

Table 6.4.6: Safe Working Stresses of Bamboos for Structural Designing⁽¹⁾

Sl No.	Species	Extreme Fibre Stress in Bending N/mm ²	Modulus of Elasticity 10 ³ N/mm ²	Allowable Compressive Stress N/mm ²
(1)	(2)	(3)	(4)	(5)
GROUP A				
i)	Bambusa glaucescens (syn. B. nana)	20.7	3.28	15.4
ii)	Dendrocalamus strictus	18.4	2.66	10.3
iii)	Oxytenanthera abyssinica	20.9	3.31	13.3
GROUP B				
iv)	Bambusa balcooa	16.05	1.62	13.3
v)	B. pallida	13.8	2.87	15.4
vi)	B. nutans	13.2	1.47	13.0
vii)	B. tulda	13.3	1.77	11.6
viii)	B. auriculata	16.3	3.34	10.5
ix)	B. burmanica	14.9	2.45	11.4
x)	Cephalostachyum pergracile	13.2	2.48	10.5
xi)	Melocanna baccifera (Syn. M. bambusoides)	13.3	2.53	15.4
xii)	Thyrsotachys oliveri	15.5	2.16	13.4
GROUP C				
xiii)	Bambusa arundinacea (Syn. B. bambos)	14.6	1.32	10.1
xiv)	B. polymorpha	9.15	1.71	8.97

xv)	B. ventricosa	8.5	0.75	10.3
xvi)	B. vulgaris	10.4	0.64	11.0
xvii)	Dendrocalamus longispathus	8.3	1.22	12.0
xviii)	Oxytenanthera nigrociliata	10.18	2.6	7.2

NOTE — The values of stress in N/mm^2 have been obtained by converting the values in kgf/cm^2 by dividing the same by 10.

¹⁾The values given pertain to testing of bamboo in green condition.

The characteristics of these groups are as given in Table 6.4.6.

Species of bamboo other than those listed in the Table 6.4.6 may be used, provided the basic strength characteristics are determined and found more than the limits mentioned therein. However, in the absence of testing facilities and compulsion for use of other species, and for expedient designing, allowable stresses may be arrived at by multiplying density with factors as given in Table 6.4.5.

4.4.3 Moisture Content in Bamboo

With decrease of moisture content (M) the strength of bamboo increases exponentially and bamboo has an intersection point (fibre saturation point) at around 25 percent moisture content depending upon the species. Matured culms shall be seasoned to about 20 percent moisture content before use.

Table 6.4.7 Limiting Strength Values (in Green Condition)

	Modulus of Rupture (R') N/mm^2	Modulus of Elasticity (E) in Bending 10^3 N/mm^2
(1)	(2)	(3)
Group A	$R' > 70$	$E > 9$
Group B	$70 \geq R' > 50$	$9 \geq E > 6$
Group C	$50 \geq R' > 30$	$6 \geq E > 3$

Table 6.4.8 Allowable Long-Term Stress (N/mm^2) per Unit Density (kg/m^3)

Condition	Axial Compression (no buckling)	Bending	Shear
Green	0.011	0.015	—
Air dry (12%)	0.013	0.020	0.003

NOTE — In the laboratory regime, the density of bamboo is conveniently determined. Having known the density of any species of bamboo, permissible stresses can be worked out using factors indicated above. For example, if green bamboo has a density of 600 kg/m^3 , the allowable stress in bending would be $0.015 \times 600 = 9 \text{ N/mm}^2$.

References: IS 6874: 1973, "Method of Test for Round Bamboo", Bureau of Indian Standards, India, 1974.

Table 6.4.5 Physical and Mechanical Properties of Bamboos (in Round Form)

SI No.	Species	Properties						
		In Green Condition				In Air Dry Conditions		
		Density kg/m ³	Modulus of Rupture N/mm ²	Modulus of Elasticity 10 ³ N/mm ²	Maximum Compressive strength N/mm ²	Density kg/m ³	Modulus of Rupture N/mm ²	Modulus of Elasticity 10 ³ N/mm ²
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	<i>Bambusa auriculata</i>	594	65.1	15.01	36.7	670	89.1	21.41
ii)	<i>B. balcooa</i>	740	64.2	7.06	38.6	850	68.3	9.12
iii)	<i>B. bambos (Syn.B.atwindinacea)</i>	559	58.3	5.95	35.3	663	80.1	8.96
iv)	<i>B. burmanica</i>	570	59.7	11.01	39.9	672	105.0	17.81
v)	<i>B. glaucescens (Syn.B.nana)</i>	691	82.8	14.77	53.9	—	—	—
vi)	<i>B. nutans</i>	603	52.9	6.62	45.6	673	52.4	10.72
vii)	<i>B. pallida</i>	731	55.2	12.90	54.0	—	—	—
viii)	<i>B. polymorpha</i>	610	36.6	6.0	31.4	840	40.6	5.89
ix)	<i>B. tulda</i>	610	53.2	10.3	39.5	830	65.8	11.18
x)	<i>B. ventricosa</i>	626	34.1	3.38	36.1	—	—	—
xi)	<i>B. vulgaris</i>	626	41.5	2.87	38.6	—	—	—
xii)	<i>Cephalostachyum pergracile</i>	601	52.6	11.16	36.7	640	71.3	19.22
xiii)	<i>Dendrocalamus giganteous</i>	597	17.2	0.61	35.2	—	—	—
xiv)	<i>D. hamiltonii</i>	515	40.0	2.49	43.4	—	—	—
xv)	<i>D. longispathus</i>	711	33.1	5.51	42.1	684	47.8	6.06
xvi)	<i>D. membranaceus</i>	551	26.3	2.44	40.5	664	37.8	3.77
xvii)	<i>D. strictus</i>	631	73.4	11.98	35.9	728	119.1	15.00
xviii)	<i>Melocanna baccifera</i>	817	53.2	11.39	53.8	751	57.6	12.93
xix)	<i>Oxytenanthera abyssinica</i>	688	83.6	14.96	46.6	—	—	—
xx)	<i>Oxytenanthera nigrociliata</i>	510	40.70	11.7	25.2	830	51.98	12.85
xxi)	<i>Thyrsostachys oliveri</i>	733	61.9	9.72	46.9	758	90.0	12.15
NOTES								

- 1 As the strength of split bamboo is more than that of round bamboo, the results of tests on round bamboo can be safely used for designing with split bamboo.
- 2 The values of stress in N/mm^2 have been obtained by converting the values in kgf/cm^2 by dividing the same by 10.

- 4.4.3.1 Air seasoning of split or half-round bamboo does not pose much problem but care has to be taken to prevent fungal discoloration and decay. However, rapid drying in open sun can control decay due to fungal and insect attack. Seasoning in round form presents considerable problem as regards mechanical degrade due to drying defects.

NOTE — A general observation has been that immature bamboo gets invariably deformed in cross-section during seasoning and thick walled immature bamboo generally collapses. Thick mature bamboo tends to crack on the surface, with the cracks originating at the nodes and at the decayed points. Moderately thick immature and thin and moderately thick mature bamboos season with much less degrade. Bamboo having poor initial condition on account of decay, borer holes, etc generally suffers more drying degrades.

- 4.4.3.2 Accelerated air seasoning method gives good results. In this method, the nodal diaphragms (septa) are punctured to enable thorough passage of hot air from one end of the resulting bamboo tube to the other end.

- References: 1. IS 6874: 1973, "Method of Test for Round Bamboo", Bureau of Indian Standards, India, 1974.
2. Salehuddin, A. B. M., Unnoto Poddhotite Bash Shongrokkhon o Babohar", Bangladesh Agriculture Research Institute, 2004.

4.4.4 Grading of Structural Bamboo

Grading is sorting out bamboo on the basis of characteristics important for structural utilization as under:

- (a) Diameter and length of culm,
- (b) Taper of culm,
- (c) Straightness of culm,
- (d) Inter nodal length,
- (e) Wall thickness,
- (f) Density and strength, and
- (g) Durability and seasoning.

One of the above characteristics or sometimes combination of 2 or 3 characteristics form the basis of grading. The culms shall be segregated species-wise.

4.4.4.1 Diameter and Length

4.4.4.1.1 Gradation according to the Mean Outer Diameter

For structural Group A and Group B species, culms shall be segregated in steps of 10 mm of mean outer diameter as follows:

- Special Grade 70mm<Diameter <100mm
- Grade I 50mm< Diameter <70mm
- Grade II 30mm Diameter <50mm
- Grade III Diameter <30mm

For structural Group C species culms shall be segregated in steps of 20 mm of mean outer diameter

- Grade I 80 mm < Diameter <100 mm
- Grade II 60 mm< Diameter< 80 mm
- Grade III Diameter <60 mm

- 4.4.4.1.2 The minimum length of culms shall be preferably 6 m for facilitating close fittings at joints.

4.4.5 Taper

The taper shall not be more than 5.8 mm per metre length (or 0.58 percent) of bamboo in any grade of bamboo.

4.4.5.1 Curvature

The maximum curvature shall not be more than 75 mm in a length of 6 m of any grade of bamboo.

4.4.5.2 Wall Thickness

Preferably minimum wall thickness of 8 mm shall be used for load bearing members.

4.4.5.3 Defects and Permissible Characteristics

4.4.5.3.1 Dead and immature bamboos, bore/GHOON holes, decay, collapse, checks more than 3 mm in depth, shall be avoided.

4.4.5.3.2 Protruded portion of the nodes shall be flushed smooth. Bamboo shall be used after at least six weeks of felling.

References: 1. IS 9096: 1979, "Code of Practice for Preservation of Bamboo for Structural Purposes", Bureau of Indian Standards, India, 1974.

4.4.5.3.3 Broken, damaged and discolored bamboo shall be rejected.

4.4.5.3.4 Matured bamboo of at least 4 years of age shall be used.

4.4.6 Durability and Treatability

4.4.6.1 Durability

The natural durability of bamboo is low and varies between 12 months and 36 months depending on the species and climatic conditions. In tropical countries the biodeterioration is very severe, Bamboos are generally destroyed in about one to two years' time when used in the open and in contact with ground while a service life of two to five years can be expected from bamboo when used under cover and out of contact with ground. The mechanical strength of bamboo deteriorates rapidly with the onset of fungal decay in the sclerenchymatous fibres. Split bamboo is more rapidly destroyed than round bamboo. For making bamboo durable, suitable treatment shall be given.

Treatability

Due to difference in the anatomical structure of bamboo as compared to timber, bamboo behaves entirely differently from wood during treatment with preservative. Bamboos are difficult to treat by normal preservation methods in dry condition and therefore treatment is best carried out in green condition.

Boucherie Process

In this process of preservative treatment, water borne preservative is applied to end surface of green bamboo through a suitable chamber and forced through the bamboo by hydrostatic or other pressure.

References: 1. Salehuddin, A. B. M., Unnoto Poddhotite Bash Shongrokkhon o Babohar", Bangladesh Agriculture Research Institute, 2004.

4.4.6.1.1 Performance of treated bamboo

Trials with treated bamboos have indicated varied durability depending upon the actual location of use. The performance in partially exposed and under covered conditions is better.

4.4.6.1.2 For provisions on safety of bamboo structures against fire, see Part 7 'Constructional Practices and Safety'.

4.5 PERMISSIBLE STRESSES

4.5.1 Factor of Safety

The safety factor for deriving stresses of bamboo shall be as under:

Extreme fibre stress in beams

4

Modulus of elasticity	4.5
Maximum compressive stress parallel to grain/fibres	3.5

4.5.2 Coefficient of Variation

The coefficient of variation (in percent) shall be as under:

Property	Mean	Range	Maximum Expected Value
(1)	(2)	(3)	(4)
Modulus of rupture	15.9	5.7-28.3	23.4
Modulus of elasticity	21.1	12.7-31.7	27.4
Maximum compressive stress	14.9	7.6-22.8	20.0

The maximum expected values of coefficient of variation which are the upper confidence limits under normality assumption such that with 97.5 percent confidence the actual strength of the bamboo culms will be at least 53 percent of the average reported value of modulus of rupture in Table 6.4..

4.5.3

Solid bamboos or bamboos whose wall thickness (w) is comparatively more and bamboos which are generally known as male bamboos having nodes very closer and growing on ridges are often considered good for structural purposes.

4.5.4

The safe working stresses for 16 species of bamboos are given in Table 6.4..

4.5.5

For change in duration of load other than continuous (long-term), the permissible stresses given in Table 6.4. shall be multiplied by the modification factors given below:

For imposed or medium term loading	1.25
For short-term loading	1.50

4.6 DESIGN CONSIDERATIONS

4.6.1

All structural members, assemblies or framework in a building shall be capable of sustaining, without exceeding the limits of stress specified, the worst combination of all loadings. A fundamental aspect of design will be to determine the forces to which the structure/structural element might be subjected to, starting from the roof and working down to the soil by transferring the forces through various components and connections. Accepted principles of mechanics for analysis and specified design procedures shall be applied (see Part 6 'Structural Design', Chapter 11 Timber Structures').

4.6.2

Unlike timber, bamboo properties do not relate well to species, being dependent among other factors, on position of the culm, geographic location and age. The practice in timber engineering is to base designs on safe working stresses and the same may be adopted to bamboo with the limitations that practical experience rather than precise calculations generally govern the detailing.

4.6.3 Net Section

It is determined by passing a plane or a series of connected planes transversely through the members. Least net sectional area is used for calculating load carrying capacity of a member.

4.6.4 Loads

The loads shall be in accordance with Part 6 'Structural Design', Chapter 2 'Loads'.

4.6.5 Structural Forms

4.6.5.1 Main structural components in bamboo may include roof and floor diaphragms, shear walls, wall panellings, beams, piles, columns, etc. Both from the point of view of capacity and deformation, trusses and framed skeltons are much better applications of bamboo.

4.6.5.2 Schematization of Bamboo as a Structural Material

This shall be based on the principles of engineering mechanics involving the following assumptions and practices:

- (a) The elastic behaviour of bamboo, till failure; (plastic behaviour being considered insignificant);
- (b) Bamboo culms are analysed on mean wall thickness basis as hollow tube structure (not perfectly straight) member on mean diameter basis:
- (c) The structural elements of bamboo shall be appropriately supported near the nodes of culm as and where the structural system demands. The joints in the design shall be located near nodes; and
- (d) Bamboo structures be designed like any other conventional structural elements taking care of details with regards to supports and joints; they shall be considered to generally act as a hinge, unless substantiating data justify a fixed joint.

4.6.6 Flexural Memders

4.6.6.1 All flexural members maybe designed using the principles of beam theory.

4.6.6.2 The tendency of bamboo beams to acquire a large deflection under long continuous loadings due to possible plastic flow, if any shall be taken care of. Permanent load may be doubled for calculation of deflection under sustained load (including creep) in case of green bamboo having moisture content exceeding 15 percent.

4.6.6.3 The moment of inertia, I shall be determined as follows:

- (a) The outside diameter and the wall thickness should be measured at both ends, correct up to 1 mm for diameter of culm and 0.1 mm for the wall thickness. (For each cross-section the diameter shall be taken twice, in direction perpendicular to each other and so the wall thickness shall be taken as four times, in the same places as the diameter has been taken twice.)
- (b) With these values the mean diameter and the mean thickness for the middle of the beam shall be calculated and moment of inertia determined.

4.6.6.4 The maximum bending stress shall be calculated and compared with the allowable stress.

4.6.6.5 For shear checks, conventional design procedure in accordance with Part 6 'Structural Design', Chapter 11 'Timber Structures' shall be followed.

NOTE— The basic shear stress values (N/mm²) for five species of bamboo in split form in green condition can be assumed as under:

Bambusa pallida	9.77
B. Vulgaris	9.44
Dedroculumus giganteous	8.86

D.humiltonii	7.77
Oxytenanthera abyssinica	11.2

- 4.6.6.6 Forces acting on a beam, being loads or reaction forces at supports, shall act in nodes or as near to nodes as by any means possible.

4.6.7 Bamboo Column (Predominantly Loaded in Axial Direction)

- 4.6.7.1 Columns and struts are essential components sustaining compressive forces in a structure. They transfer load to the supporting media.

- 4.6.7.2 Design of columns shall be based on one of the following two criteria:

- (a) Full scale buckling tests on the same species, size and other relevant variables.
- (b) Calculations, based on the following:
 - i) The moment of inertia shall be as per 6.6.3.
 - ii) For bamboo columns the best available straight bamboo culms shall be selected. Structural bamboo components in compression should be kept under a slenderness ratio of 50.
 - iii) The bending stresses due to initial curvature, eccentricities and induced deflection shall be taken into account, in addition to those due to any lateral load.

- 4.6.7.3 Buckling calculation shall be according to Euler, with a reduction to 90 percent of moment of inertia, to take into account the effect of the taper, provided the reduced diameter is not less than 0.6 percent.

- 4.6.7.4 For strength and stability, larger diameter thick walled sections of bamboo with closely spaced nodes shall be used. Alternatively, smaller sections may be tied together as a bundle-column.

4.6.8 Assemblies, Roof Trusses

- 4.6.8.1 A truss is essentially a plane structure which is very stiff in the plane of the members, that is the plane in which it is expected to carry load, but very flexible in every other direction. Roof truss generally consists of a number of triangulated frames, the members of which are fastened at ends and the nature of stresses at joints are either tensile or compressive and designed as pin-ended joints (see Fig. 6.4.1.(a)). Bamboo trusses may also be formed using bamboo mat board or bamboo mat-veneer composite or plywood gusset (see Fig. 6.4.1.(b)).

- 4.6.8.2 Truss shall be analysed from principles of structural mechanics for the determination of axial forces in members. For the influence of eccentricities, due allowance shall be made in design.

- 4.6.8.3 The truss height shall exceed 0.15 times the span in case of a triangular truss (pitched roofing) and 0.10 times the span in case of a rectangular (parallel) truss.

- 4.6.8.4 For members in compression, the effective length for in-plane strength verification shall be taken as the distance between two adjacent points of contraflexure. For fully triangulated trusses, effective length for simple span members without especially rigid end-connection shall be taken as the span length.

- 4.6.8.5 For strength verification of members in compression and connections, the calculated axial forces should be increased by 10 percent.

- 4.6.8.6 The spacing of trusses shall be consistent with use of bamboo purlins (2 m to 3 m).

- 4.6.8.7 The ends in open beams, joists, rafters, purlins shall be suitably plugged. Bamboo roof coverings shall be considered as non-structural in function. The common roof covering shall include bamboo mat board, bamboo mat corrugated sheet, bamboo tiles/ strings, plastered bamboo reeds, thatch, corrugated galvanized iron sheeting, plain clay tiles or pan tiles, etc.

4.7 DESIGN AND TECHNIQUES OF JOINTS

4.7.1 Bamboo Joints

Round, tubular form of bamboo requires an approach different to that used for sawn timber. Susceptibility to crushing at the open ends, splitting tendency, variation in diameter, wall thickness and straightness are some of the associated issues which have to be taken care of while designing and detailing the connections with bamboo.

4.7.1.1 Traditional Practices

Such joining methods revolve around lashing or tying by rope or string with or without pegs or dowels. Such joints lack stiffness and have low efficiency.

4.7.1.1.1 Lengthening Joints (End Joints)

Lap Joint

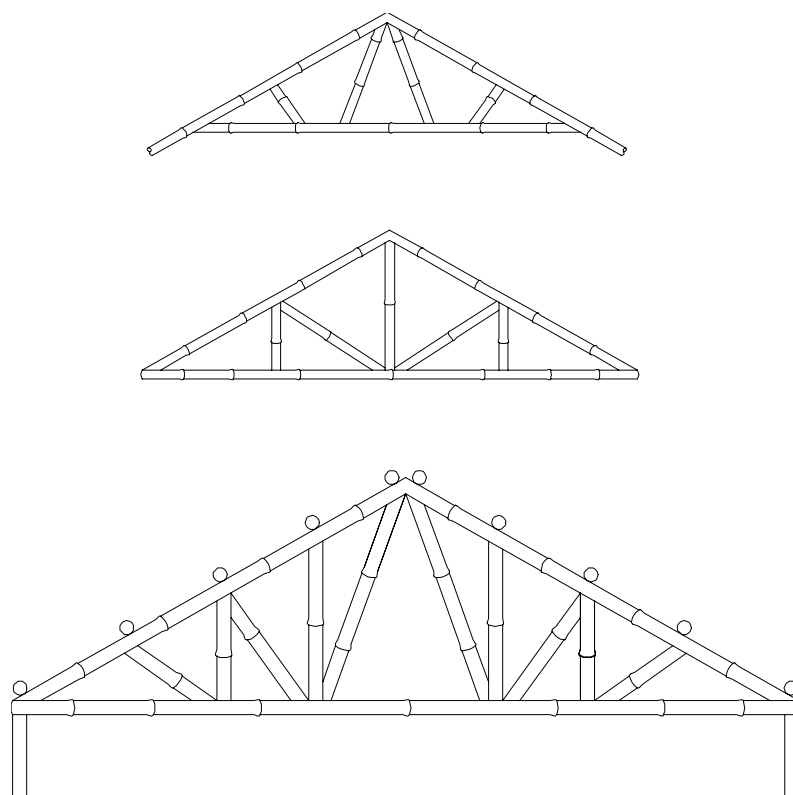
In this case, end of one piece of bamboo is made to lap over that of the other in line and the whole is suitably fastened. It maybe full lapping or half lapping. Full section culms are overlapped by at least one internode and tied together in two or three places. Efficiency could be improved by using bamboo or hardwood dowels. In half lapping, culms shall preferably be of similar diameter and cut longitudinally to half depth over at least one internode length and fastened as per full lap joint (see Fig. 6.4.2).

Butt Joint

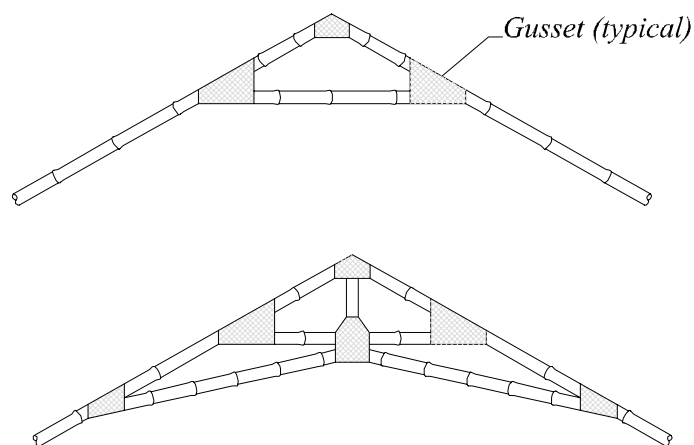
Culms of similar diameter are butted end to end, interconnected by means of side plates made of quarterround culm of slightly large diameter bamboo, for two or more internode lengths. Assembly shall be fixed and tied preferably with dowel pins. This joint transfers both compressive and tensile forces equally well (see Fig. 6.4.3).

Sleeves and Inserts

Short length of bamboo of appropriate diameter may be used either externally or internally to join two culms together (see Fig. 6.4.4).



6.4.1(a)- Pin ended joint trusses



6.4.1(b)- Gusset joint trusses

Fig. 6.4.1 Some typical configurations for small and large trusses in Bamboo

Scarf Joint

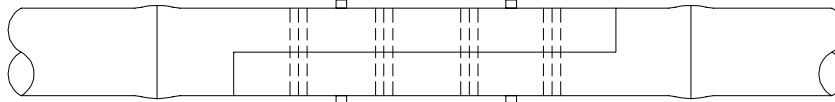
A scarf joint is formed by cutting a sloping plane 1 in 4 to 6 on opposite sides from the ends of two similar diameter bamboo culms to be joined. They shall be lapped to form a continuous piece and the assembly suitably fastened by means of lashings. Using hooked splays adds to the strength and proper location of joints (see Fig. 6.4.5).

4.7.1.1.2 Bearing Joints

For members which either bear against the other or cross each other and transfer the loads at an angle other than parallel to the axis, bearing joints are formed.

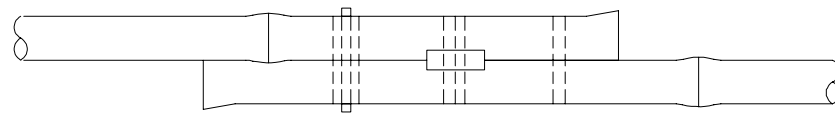
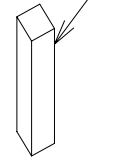


6.4.2.(a)- Full-Lapped spliced joint
(Over - Lapped atleast one internode)



6.4.2.(b)- Full-Lapped spliced joint
(Similar diameter; One internode length)

Bamboo or
Hardwood dowel



6.4.2(c) Lapped spliced joint with pegs and battens

Fig. 6.4.2 Lap joint in Bamboo

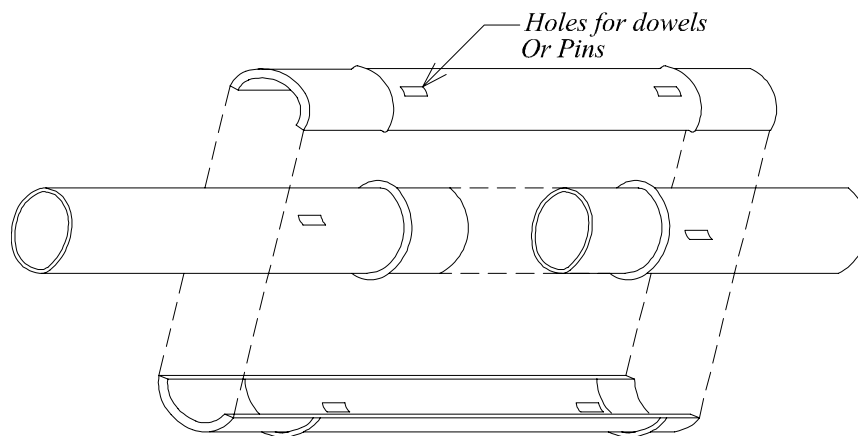


Fig: 6.4.3 Butt joint with side plates in Bamboo

Butt Joint

The simplest form consists of a horizontal member supported directly on top of a vertical member. The top of the post may be cut to form a saddle to ensure proper seating of beam for good load transfer. The saddle should be close to a node to reduce risk of splitting (see Fig. 6.4.6).

Tenon Joint

It is formed by cutting a projection (tenon) in walls of one piece of bamboo and filling it into corresponding holes (mortise) in another and keyed. It is a neat and versatile joint for maximum strength and resistance to separation (see Fig. 6.4.7).

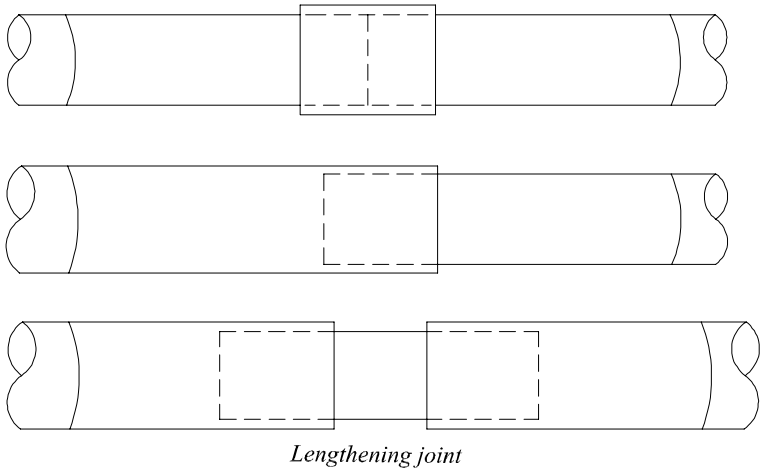


Fig:6.4.4 Sleeves and inserts for Bamboo joint

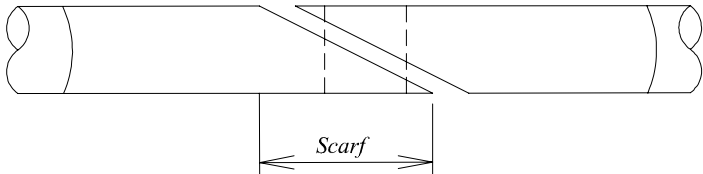
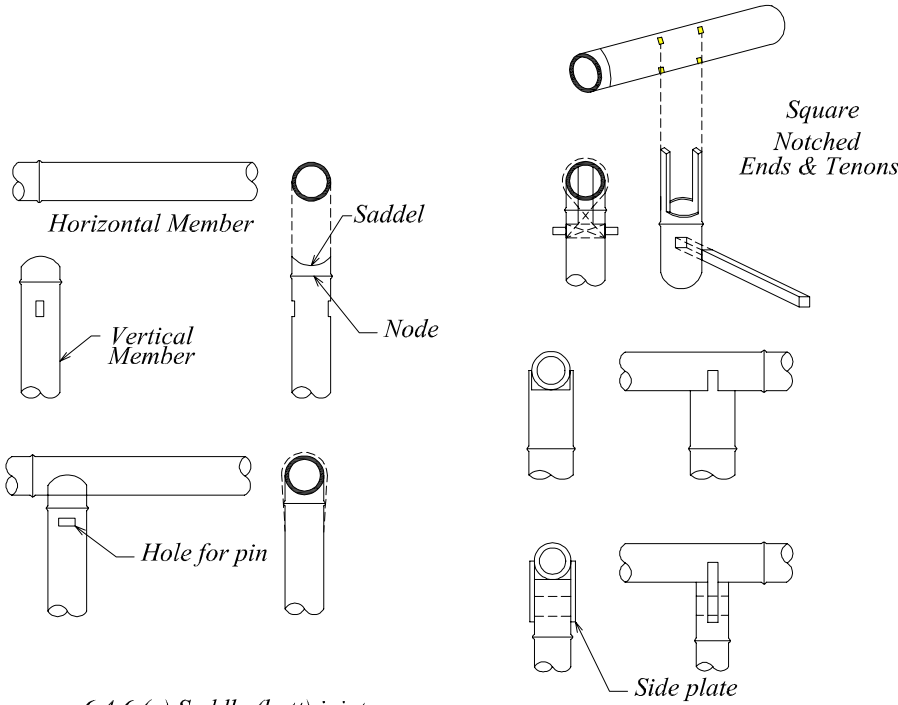


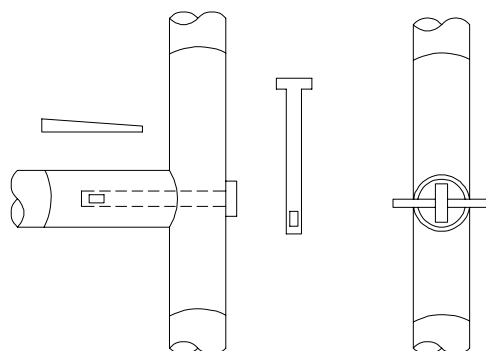
Fig: 6.4.5 Scarf joint



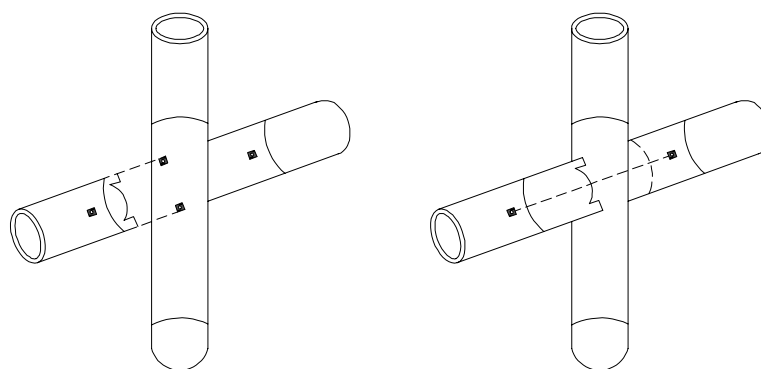
6.4.6.(a) Saddle (butt) joint
(Saddel to be close to the node)

6.4.6(b) Variations of saddle joint

Fig: 6.4.6 Butt joints in Bamboo



Tenon and key joint



Integral tenon (horned) joint

Fig: 6.4.7 Tenon joint

Cross-Over Joint

It is formed when two or more members cross at right angles and its function is to locate the members and to provide lateral stability. In case of the joint connecting floor beam to post, it may be load bearing (see Fig. 6.4.8). Such joints are also used to transmit angle thrust.

Angled Joint

When two or more members meet or cross other than at right angles, angled joints are formed. For butt joints, the ends of the members may be shaped to fit in as saddle joints. Tenons would help in strengthening such joints (see Fig. 6.4.9).

4.7.1.2 Modern Practices

Following are some of the modern practices for bamboo jointing (see Fig. 6.4.10):

- (a) Plywood or solid timber gusset plates may be used at joint assemblies of web and chord connection in a truss and fixed with bamboo pins or bolts. Hollow cavities of bamboo need to be stuffed with wooden plugs.
- (b) Use of wooden inserts to reinforce the ends of the bamboo before forming the joints. Alternatively steel bands clamps with integral bolt/eye may be fitted around bamboo sections for jointing.

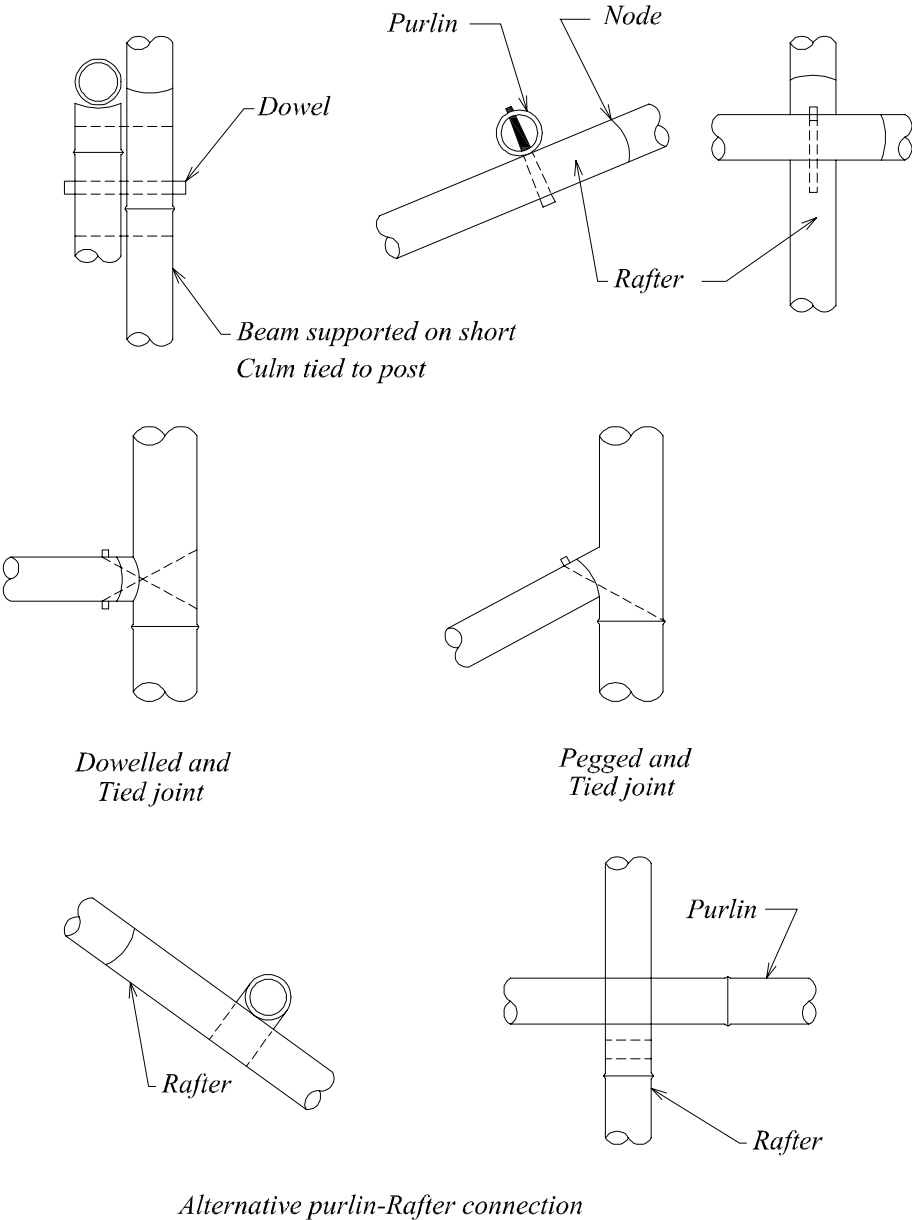


Fig: 6.4.8 Cross over joints (Bearing joints)

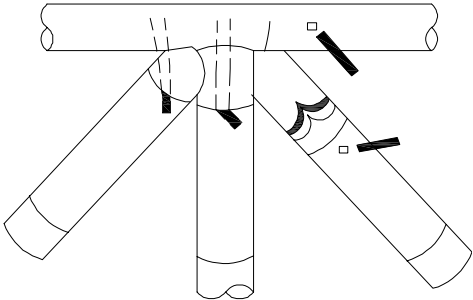


Fig: 6.4.9 Angled joints with integral tenons

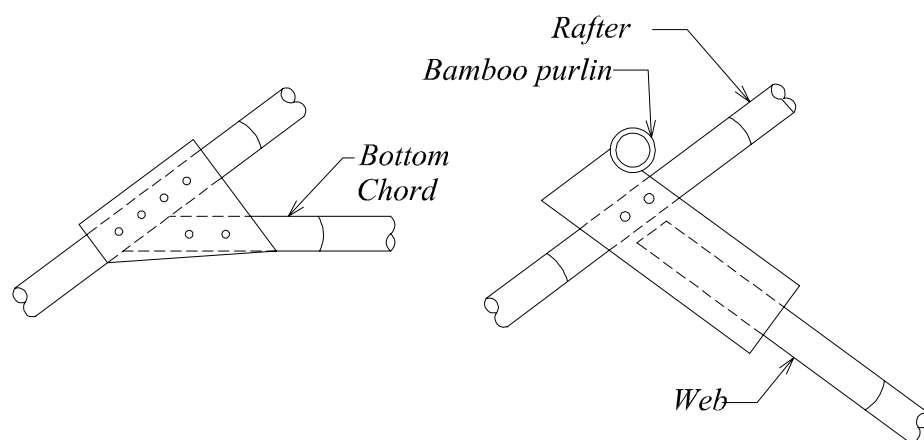


Fig: 6.4.10 Gusset plated joint

4.7.1.3 Fixing Methods and Fastening Devices

In case of butt joints the tie maybe passed through a pre-drilled hole or around hardwood or bamboo pegs or dowels inserted into prefomed holes to act as horns. Pegs are driven from one side, usually at an angle to increase strength and dowels pass right through the member, usually at right angles.

4.7.1.3.1 Normally 1.60 mm diameter galvanized iron wire may be used for tight lashing.

4.7.1.3.2 Wire Bound Joints

Usually galvanized iron 2.00 mm diameter galvanized iron wire is tightened around the joints by binding the respective pieces together. At least two holes are drilled in each piece and wire is passed through them for good results.

4.7.1.3.3 Pin And Wire Bound Joints

Generally 12 mm dia bamboo pins are fastened to culms and bound by 2.00 mm diameter galvanized iron wire.

4.7.1.3.4 Fish Plates/Gusset Plated Joints

At least 25 mm thick hardwood splice plate or 12 mm thick structural grade plywood are used. Solid bamboo pins help in fastening the assembly.

4.7.1.3.5 Horned Joints

Two tongues made atone end of culm may be fastened with across member with its mortise grooves to receive horns, the assembly being wire bound.

4.7.1.4 For any complete joint alternative for a given load and geometry, description of all fastening elements, their sizes and location shall be indicated. Data shall be based on full scale tests.

4.7.1.5 Tests on full scale joints or on components shall be carried out in a recognized laboratory.

4.7.1.6 In disaster high wind and seismic areas, good construction practice shall be followed taking care of joints, their damping and possible ductility. Bracings in walls shall be taken care of in bamboo structures.

4.8 STORAGE OF BAMBOO

Procurement and storage of bamboo stocks are essential for any project work and shall be done in accordance with Part 7 'Constructional Practices and Safety'.

Chapter 5

CONCRETE MATERIAL

5.1 GENERAL

5.1.1 Scope

The provisions of this chapter shall apply to the design of reinforced and prestressed concrete structures specified in chapters 6, 8, 9 shall be applicable for normal weight aggregate only unless otherwise specified.

5.1.2 Notation

- C_c = creep coefficient
- E_c = modulus of elasticity of concrete
- E_s = modulus of elasticity of reinforcement
- E_t = modulus of elasticity of concrete at the age of loading t
- f'_c = specified compressive strength of concrete
- f'_{cr} = required average compressive strength of concrete used
as the basis for selection of concrete proportions
- f_y = specified yield strength of reinforcement
- K = coefficient of shrinkage
- s = standard deviation
- w_c = unit weight of concrete
- ϵ_{cc} = creep strain in concrete
- ϵ_{sh} = shrinkage of plain concrete
- ρ = area of steel relative to that of the concrete.

5.2 CONSTITUENTS OF CONCRETE

5.2.1 Cement

5.2.1.1 Cement shall conform to one of the following specifications :

- (a) "BDS EN 197-1:2003
- (b) "Specification for Portland Cement" (ASTM C150 / C150M - 09)
- (c) "Specification for Blended Hydraulic Cement" (ASTM C595 / C595M - 10),
- (d) " ASTM C1157 / C1157M - 10 "

5.2.1.2 Cement used in the construction shall be the same as that used in the concrete mix design.

5.2.2 Aggregates

5.2.2.1 Concrete aggregates shall conform to the standards "Coarse and Fine Aggregates from Natural Sources for Concrete" (BDS 243 : 1963); "Specification for Concrete Aggregates" (ASTM C33 / C33M - 08).

5.2.2.2 Maximum nominal size of coarse aggregate shall be the minimum of the following :

- (a) $\frac{1}{5}$ the narrowest dimension between sides of forms,

- (b) $\frac{1}{3}$ the depth of slabs,
- (c) $\frac{3}{4}$ the minimum clear spacing between individual reinforcing bars, or bundles of bars, or prestressing tendons or ducts.

The above limitations may be relaxed if, in the judgment of the engineer, workability and methods of consolidation are such that concrete can be placed without honeycomb or voids.

- 5.2.2.3 Coarse aggregate made from Grade A bricks as specified in BDS 208 "Specification for Common Building Clay Bricks" may be used in different types slab and non structural elements, except in applications where the ambient environmental conditions may impair the performance of concrete made of such aggregates.

5.2.3 Water

- 5.2.3.1 Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be harmful to concrete or reinforcement.
- 5.2.3.2 For concrete wherein aluminium members will be embedded, mixing water shall not contain harmful amounts of chloride ion as indicated in Sec 5.5.3.
- 5.2.3.3 Nonpotable water shall not be used in concrete except the following conditions:
- (a) Selection of concrete proportions shall be based on concrete mixes using water from the same source.
 - (b) Nonpotable water is permitted only if specified comparative mortar test cubes made with nonpotable water produce at least 90 per cent of the strength achieved with potable water.

5.2.4 Admixtures

- 5.2.4.1 Prior approval of the engineer shall be required for the use of admixtures in concrete. All admixtures shall conform to the requirements of this section and Sec 2.4.5 of Part 5.
- 5.2.4.2 Admixture used in the work shall be the same as that used in the concrete mix design.
- 5.2.4.3 Admixtures containing chloride other than impurities from admixture ingredients shall not be used in concrete containing embedded aluminium, or in concrete cast against permanent galvanized metal forms (see Sec 5.5.1.2 and 5.5.2.1).
- 5.2.4.4 Air entraining admixtures, if used in concrete, shall conform to "Specification for Air entraining Admixtures for Concrete" (ASTM C260 - 06).
- 5.2.4.5 Water reducing admixtures, retarding admixtures, accelerating admixtures, water reducing and retarding admixtures, and water reducing and accelerating admixtures, if used in concrete, shall conform to "Specification for Chemical Admixtures for Concrete" (ASTM C494 / C494M - 10) or "Specification for Chemical Admixtures for use in Producing Flowing Concrete" (ASTM C1017 / C1017M - 07).
- 5.2.4.6 Fly ash or other pozzolans used as 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 C618 - 08a).
- 5.2.4.7 Ground granulated blast-furnace slag used as an admixture shall conform to "Specification for Ground Iron Blast Furnace Slag for use in Concrete and Mortar" (ASTM C989 - 09a).

5.3 STEEL REINFORCEMENT

5.3.1 General

- 5.3.1.1 Steel reinforcement for concrete shall conform to the provisions of this section and those of Sec 2.4.6 of Part 5.
- 5.3.1.2 Modulus of elasticity E_s for reinforcement shall be taken as 200 kN/mm².
- 5.3.1.3 Reinforcing bars to be welded shall be indicated on the drawings and welding procedure to be used shall be specified. Reinforcing bars otherwise conforming to BDS ISO 6935-2:2006, shall also possess material properties necessary to conform to welding procedures specified in "Structural Welding Code - Reinforcing Steel" (AWS D1.4) of the American Welding Society.

5.3.2 Deformed Reinforcement

- 5.3.2.1 Deformed reinforcing bars shall conform to one of the following specifications:
 - (a) "Specification for Steel Bars and Wires for the Reinforcement of Concrete" (BDS 1313),
 - (b) "Specification for Deformed and Plain Billet Steel Bars for Concrete Reinforcement" (ASTM A615 / A615M - 09b),
 - (c) "Specification for Rail Steel Deformed and Plain Bars for Concrete Reinforcement" Including Supplementary Requirements S1 (ASTM A996 / A996M - 09b including S1),
 - (d) "Specification for Axle Steel Deformed and Plain Bars for Concrete Reinforcement" (ASTM A996 / A996M - 09b),
 - (e) "Specification for Low Alloy Steel Deformed Bars for Concrete Reinforcement" (ASTM A706 / A706M - 09b),
 - (f) "Specification for Cold Worked Steel Bars for the Reinforcement of Concrete" (BS 4461).
- 5.3.2.2 Deformed reinforcing bars with a specified yield strength f_y exceeding 410 N/mm² shall be permitted, 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 Sec 5.3.2.1 (Also see Sec 6.1.2.5).
- 5.3.2.3 Galvanized reinforcing bars shall comply with "Specification for Zinc Coated (Galvanized) Steel Bars for Concrete Reinforcement " (ASTM A767 / A767M - 09). Epoxy coated reinforcing bars shall comply with "Specifications for Epoxy Coated Reinforcing Steel Bars" (ASTM A775 / A775M - 07b). Galvanized or epoxy coated reinforcement shall also conform to one of the standards listed in Sec 5.3.2.1 above.

5.3.3 Plain Reinforcement

- 5.3.3.1 Plain bars shall conform to one of the specifications listed in Section 5.3.2.1 (a), (b), (c) or (d).
- 5.3.3.2 Plain wire shall conform to "Specification for Steel Wire, Plain, for Concrete Reinforcement" (ASTM A82 / A82M - 07) except that for wire with a specified yield strength f_y exceeding 410 N/mm², f_y shall be the stress corresponding to a strain of 0.0035.
- 5.3.3.3 Plain bars and wire may be used as ties, stirrups and spirals for all structural members and for all reinforcement in structures up to 4-storey high.

5.3.4 Structural Steel, Steel Pipe or Tubing

- 5.3.4.1 Structural steel used with reinforcing bars in composite compression members meeting the requirements of Sec 6.3.10.8 or 6.3.10.9 shall conform to one of the following specifications:
 - (a) "Specification for Structural Steel" (ASTM A36 / A36M - 08),

- (b) "Specification for High Strength Low Alloy Structural Steel"(ASTM A242 / A242M - 04(2009)),
- (c) "Specification for High Strength Low Alloy Structural Manganese Vanadium Steel" (ASTM A572 / A572M - 07),
- (d) "Specification for High Strength Low Alloy Columbium-Vanadium Steels of Structural Quality" (ASTM A572 / A572M - 07),
- (e) "Specification of High Strength Low Alloy Structural Steel with 50 ksi (345 Mpa) Minimum Yield Point to 4 in (100 mm) Thick" (ASTM A588 / A588M - 05).

5.3.4.2 Steel pipe or tubing for composite compression members composed of a steel encased concrete core meeting the requirements of Sec 6.3.10.7 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 A53 / A53M - 07).
- (b) "Specification for Cold Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes" (ASTM A500 / A500M - 10).
- (c) "Specification for Hot Formed Welded and Seamless Carbon Steel Structural Tubing " (ASTM A501 - 07).

5.4 WORKABILITY OF CONCRETE

Concrete mix proportions shall be such that the concrete is of adequate workability and can properly be compacted. Suggested ranges of values of workability of concrete for some placing conditions, are given in Table 6.5.1.

Table 6.5.1: Suggested Workability of Concrete for Various Placing Conditions

Placing Conditions	Degree of Workability	Values of Workability
Concreting of thin sections with vibration	Very low	20-10 seconds Vee-Bee time or 0.75-0.80 compacting factor
Concreting of lightly reinforced sections with vibration	Low	10-5 seconds Vee-Bee time or 0.80-0.85 compacting factor
Concreting of lightly reinforced sections without vibration or heavily reinforced section with vibration	Medium	5-2 seconds Vee-Bee time or 0.85-0.92 compacting factor or 25-75 mm slump for 20* mm aggregate
Concreting of heavily reinforced sections without vibration	High	Above 0.92 compacting factor or 75-125 mm slump for 20* mm aggregate
* For smaller aggregates the values will be lower		

5.5 DURABILITY OF CONCRETE

5.5.1 Special Exposures

- 5.5.1.1 For concrete intended to have low permeability when exposed to water, the water cement ratio shall not exceed 0.50.
- 5.5.1.2 For corrosion protection of reinforced concrete exposed to brackish water, sea water or spray from these sources, the water cement ratio shall not exceed 0.4.
If minimum concrete cover required by Sec 8.1.8 is increased by 12 mm, water cement ratio may be increased to 0.45.
- 5.5.1.3 The water cement ratio required in Sec 5.5.1.1 and 5.5.1.2 above and Table 6.5.2 shall be calculated using the weight of cement meeting the requirements of BDS EN-197-1 or ASTM C595 / C595M - 10 or C1157 / C1157M - 10, plus the weight of fly ash or pozzolan satisfying ASTM C618 - 08a and/or slag satisfying ASTM C989 - 09a, if any.

5.5.2 Sulphate Exposures

- 5.5.2.1 Concrete to be exposed to sulphate containing solutions or soils shall conform to the requirements of Table 6.5.2 or be made with a cement that provides sulphate resistance with the maximum water cement ratio provided in Table 6.5.2.
- 5.5.2.2 Calcium chloride shall not be used as an admixture in concrete exposed to severe or very severe sulphate containing solutions, as defined in Table 6.5.2.

5.5.3 Corrosion of Reinforcement

- 5.5.3.1 For corrosion protection, maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures, shall not exceed the limits of Table 6.5.3. When testing is performed to determine water soluble chloride ion content, test procedure shall conform to AASHTO T260, "Methods of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials".
- 5.5.3.2 When reinforced concrete will be exposed to brackish water, sea water, or spray from these sources, requirements of Sec 5.5.1.1 and 5.5.1.2 for water cement ratio, or concrete strength and minimum cover requirements of Sec 8.1.8 shall be satisfied.

5.5.4 Minimum Concrete Strength

Minimum concrete strength for structural use of reinforced concrete shall be 20 N/mm². However, for buildings up to 4 storey, the minimum concrete strength may be relaxed to 17 N/mm².

Table 6.5.2 : Requirements for Normal Weight Aggregate Concrete Exposed to Sulphate Containing Solutions

Sulphate Exposure	Water Soluble Sulphate (SO ₄) in Soil, per cent by Weight	Sulphate (SO ₄) in Water, (ppm)	Cement Type ¹	Maximum Water Cement Ratio, by Weight
Negligible	0.00-0.10	0 – 150	-	-

Moderate ²	0.10-0.20	150 -1500	Other than CEM I and B type	0.50
Severe	0.20-2.00	1500 - 10,000	Other than CEM-I and B type	0.45
Very severe	Over 2.00	Over 10,000	Other than CEM-I and B type	0.45

Note : ¹ For types of cement see ASTM C 150 and C 595

² Sea water

³ Pozzolan that has been determined by test or service record to improve sulphate resistance when used in concrete containing Type V cement.

Table 6.5.3: Maximum Chloride Ion Content for Corrosion Protection

Type of Member	Maximum Water Soluble Chloride Ion (Cl-) in Concrete, Per Cent by Weight of Cement
Prestressed 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

5.6 CONCRETE MIX PROPORTION

5.6.1 General

5.6.1.1 Proportions of materials for concrete shall be such that :

- (a) Workability and consistency are achieved for proper placement into forms and around reinforcement, without segregation or excessive bleeding;
- (b) Resistance to special exposures to meet the durability requirements of Sec 5.5 are provided; and
- (c) Conformance with strength test requirements of Sec 5.12 is ensured.

5.6.1.2 Where different materials are to be used for different portions of the proposed work, each combination shall be evaluated.

5.6.1.3 Concrete proportions, including water cement ratio, shall be established on the basis of field experience and/or trial mixtures with materials to be employed (Sec 5.6.2) except as permitted in Sec 5.6.3 or required by Sec 5.5.

5.6.2

Proportioning Concrete Mix on the Basis of Field Experience and/or Trial Mixtures

5.6.2.1 Standard Deviation

- (a) A standard deviation shall be established where test records are available in a concrete production facility. Test records from which a standard deviation is calculated shall meet the following requirements :
- These shall represent materials, quality control procedures, and conditions similar to those expected for the proposed work. Deviations in materials and proportions for the proposed work shall be more restricted than those within the test records.
 - Test records shall represent concrete produced to meet a specified strength f'_c within 7 N/mm² of that specified for the proposed work.
 - The record shall consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in Sec 5.12.2.4 except as provided in (b) below.
- (b) Where a concrete production facility does not have test records meeting the requirements of (a) above but does have a record based on 15 to 29 consecutive tests, a standard deviation shall be established as the product of the calculated standard deviation and the modification factor specified in Table 6.5.4. However, the test records shall meet the requirements (i) and (ii) of (a) above and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

Table 6.5.4 : Modification Factor for Standard Deviation when Less Than 30 Tests are Available

No. of Tests*	Modification Factor for Standard Deviation **
Less than 15	see Sec 5.6.2.2(b)
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 the required average strength f'_{cr} from 5.6.2.2(a).

5.6.2.2 Required Average Strength

- (a) Required average compressive strength f'_c used as the basis for selection of concrete proportions shall be the larger of the values given by Eq (5.6.1) and (5.6.2) using a standard deviation calculated in accordance with Sec 5.6.2.1(a) or 5.6.2.1(b) above.

$$f'_{cr} = f'_c + 1.34s \quad (5.6.1)$$

$$f'_{cr} = f'_c + 2.33s - 3.5 \quad (5.6.2)$$

- (b) When a concrete production facility does not have field strength test records for calculation of standard deviation meeting the requirements of Sec 5.6.2.1(a) or 5.6.2.1(b), the required average strength shall be determined from Table 6.5.5 and documentation of the average strength shall be in accordance with the requirements of Sec 5.6.2.3 below.

Table 6.5.5 : Required Average Compressive Strength when Data are not Available to Establish a Standard Deviation

Specified Compressive Strength f'_c N/mm ²	Required Average Compressive Strength, f'_{cr} N/mm ²

Less than 20	$f'_{c+7.0}$
20 to 35	$f'_c + 8.5$
Over 35	$f'_{c+10.0}$

5.6.2.3 Documentation of Average Strength

Documentation shall be prepared to demonstrate that the proposed concrete proportions will produce an average compressive strength equal to or greater than the required average compressive strength (Sec 5.6.2.2). Such documentation shall consist of one or more field strength test records or trial mixtures.

- (a) When test records are used to demonstrate that proposed concrete proportions will produce the required average strength f'_{cr} (Sec 5.6.2.2) such records shall represent materials and conditions similar to those expected. Deviations 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 are acceptable provided the test records encompass a period of time not less than 45 days. Required concrete proportions shall be permitted to be established by interpolation between the strengths and proportions of two or more test records each of which meets other requirements of this section.
- (b) When an acceptable record of field test results is not available, concrete proportions may be established based on trial mixtures meeting the following restrictions :
 - i) Combination of materials shall be those for the proposed work.
 - ii) Trial mixtures having proportions and consistencies required for the 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 .
 - iii) Trial mixtures shall be designed to produce a slump within ± 20 mm of the maximum permitted, and for air entrained concrete the air content shall be within ± 0.5 per cent of the maximum allowable.
 - iv) 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 C192 / C192M - 07). Cylinders shall be tested at 28 days or at test age designated for the determination of f'_c .
 - v) From the results of cylinder tests, a curve shall be plotted showing the relationship between the water cement ratio or cement content and the compressive strength at designated test age.
 - vi) Maximum water cement ratio or minimum cement content for concrete to be used in the proposed work shall be that shown by the above curve to produce the average strength required by Sec 5.6.2.2 unless a lower water cement ratio or higher strength is required by Sec 5.5.

5.6.3 Proportioning by Water Cement Ratio

- 5.6.3.1 If the data required in Sec 5.6.2 are not available, concrete proportions shall be based on water cement ratio limits specified in Table 6.5.6 when approved by the engineer.

Table 6.5.6 : Maximum Permissible Water Cement Ratios for Concrete when Strength Data from Field Experience or Trail Mixers are not Available

Specified Compressive	
Strength*, f'_c	Absolute Water Cement Ratio by Weight

N/mm ²	Concrete other than air-entrained	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 that required in Sec 5.6.2.2.

** For strengths above 30 N/mm² (25 N/mm² for air entrained concrete) concrete proportions shall be established by methods of Sec 5.6.2.

5.6.3.2 Table 6.5.6 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 "Specification for Portland Cement" (ASTM C150 / C150M - 09), 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 "Specification for Blended Hydraulic Cements" (ASTM C595 / C595M - 10), and shall not be applied to concrete containing lightweight aggregates or admixtures other than those for entraining air.

5.6.3.3 Concrete proportioned by water cement ratio limits prescribed in Table 6.5.6 shall also conform to special exposure requirements of Sec 5.5 and to compressive strength test criteria of Sec 5.12.

5.6.4 Average Strength Reduction

As data become available during construction, amount by which value of f'_c must exceed specified value of f'_c may be reduced, provided:

- 30 or more test results are available and the average of test results exceeds that required by Sec 5.6.2.2(a) using a standard deviation calculated in accordance with Sec 5.6.2.1(a), or
- 15 to 29 test results are available and the average of test results exceeds that required by Sec 5.6.2.2(a) using a standard deviation calculated in accordance with Sec 5.6.2.1(b), and provided further that special exposure requirements of Sec 5.5 are met.

5.7 Preparation of Equipment and Place of Deposit

Preparation before concrete placement shall include the following:

- All equipment for mixing and transporting concrete shall be clean.
- All debris shall be removed from spaces to be occupied by concrete.
- Forms shall be properly cleaned and coated.
- Masonry filler units that will be in contact with concrete shall be soaked thoroughly.
- Reinforcement shall be thoroughly clean of deleterious coatings.
- Water shall be removed from place of deposit before concrete is placed unless a tremie is used or unless otherwise permitted by the engineer.
- All laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete.

5.8 MIXING

5.8.1

All concrete shall be mixed thoroughly until there is a uniform distribution of materials and shall be discharged completely before the mixer is recharged.

5.8.2

Ready mixed concrete shall be mixed and delivered in accordance with the requirements of "Specification for Ready Mixed Concrete" (ASTM C94) or "Specification for Concrete Made by Volumetric Batching and Continuous Mixing" (ASTM C685).

5.8.3

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 90 seconds 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 C94).
- (d) Materials handling, batching, and mixing shall conform to the applicable provisions of "Specification for Ready Mixed Concrete" (ASTM C94).
- (e) A detailed record shall be kept to identify:
 - i) number of batches produced;
 - ii) proportions of materials used;
 - iii) approximate location of final deposit in structure;
 - iv) time and date of mixing and placing.

5.9 CONVEYING

5.9.1

Concrete shall be conveyed from the mixer to the place of final deposit by methods that will prevent segregation or loss of materials.

5.9.2

Conveying equipment shall be capable of providing a supply of concrete to the place of deposit without segregation of ingredients and without interruptions sufficient to permit loss of plasticity between successive increments.

5.10 DEPOSITING

5.10.1

Concrete shall be deposited as near its final position as practical to avoid segregation due to rehandling or flowing.

5.10.2

Concreting shall be carried on at such a rate that concrete is at all times plastic and flows readily into spaces between and around the reinforcement.

5.10.3

Concrete that has partially hardened or been contaminated by foreign materials shall not be deposited in the structure.

5.10.4

Retempered concrete or concrete that has been remixed after initial set shall not be used.

5.10.5

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 Sec 5.16.4.

5.10.6

Top surfaces of vertically formed lifts shall be generally level.

5.10.7

When construction joints are required, joints shall be made in accordance with Sec 5.16.4.

5.10.8

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.

5.11 CURING

5.11.1

Concrete (other than high early strength) shall be maintained above 10oC and in a moist condition for at least the first 7 days after placement, except when cured in accordance with Sec 5.11.3.

5.11.2

High early strength concrete shall be maintained above 10oC and in a moist condition for at least the first 3 days, except when cured in accordance with Sec 5.11.3.

5.11.3 Accelerated Curing

5.11.3.1 Curing by high pressure steam, steam at atmospheric pressure, heat and moisture or other accepted processes, shall be permitted to accelerate strength gain and reduce time of curing.

5.11.3.2 Accelerated curing shall provide a compressive strength of the concrete at the load stage considered, at least equal to the required design strength at that load stage.

5.11.3.3 Curing process shall be such as to produce concrete with a durability at least equivalent to that obtained for concrete cured by the method of Sec 5.11.1 or 5.11.2.

5.11.4

When required by the engineer, supplementary strength tests in accordance with Sec 5.12.4 shall be performed to assure that curing is satisfactory.

5.12 EVALUATION AND ACCEPTANCE OF CONCRETE

5.12.1 General

5.12.1.1 Concrete shall be proportioned to provide an average compressive strength as prescribed in Sec 5.6.2.2 as well as to satisfy the durability criteria of Sec 5.5. Concrete shall be produced to limit frequency of strengths below f'_c to that prescribed in Sec 5.12.3.3.

5.12.1.2 Requirements of shall be based on tests of cylinders made and tested as prescribed in Sec 5.12.3.

5.12.1.3 Unless otherwise specified, f'_c shall be based on 28 day tests. Test age for f'_c shall be indicated in design drawings or specifications, if it is different from 28 days.

5.12.1.4 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

5.12.2 Frequency of Testing

5.12.2.1 Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, nor less than once for each 60 m³ of concrete, nor less than once for each 250 m² surface area for slabs or walls.

5.12.2.2 On a given project, if the total volume of concrete is such that frequency of testing required by Sec 5.12.2.1 above would provide less than three strength tests for a given class of concrete, tests shall be made from at least three randomly selected batches or from each batch if three or fewer batches are used.

5.12.2.3 When the total quantity of a given class of concrete is less than 20 m³, strength tests are not required when evidence of satisfactory strength is submitted to and approved by the engineer.

5.12.2.4 A strength test result shall be the average of the strengths of two cylinders made from the same sample of concrete and tested at 28 days or at the test age designated in accordance with Sec 5.12.1.3.

5.12.3 Laboratory Cured Specimens

5.12.3.1 Samples for strength tests shall be taken in accordance with "Method of Sampling Freshly Mixed Concrete" (ASTM C172 - 08).

5.12.3.2 Cylinders for strength tests shall be moulded and laboratory cured in accordance with "Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C31 / C31M - 09) and tested in accordance with "Test Method for Compressive Strength of Cylindrical Concrete Specimens" (ASTM C39 / C39M - 09a).

5.12.3.3 Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met :

- (a) Average of three consecutive strength tests (see Sec 5.12.2.4) equals or exceeds f'_c
- (b) No individual strength test (average of two cylinders) falls below by more than 3.5 N/mm².

5.12.3.4 If either of the requirements of Sec 5.12.3.3 are not met, steps shall be taken to increase the average of the subsequent strength test results. Requirements of Sec 5.12.5 shall be satisfied if the requirement of Sec 5.12.3.3(b) is not met.

5.12.4 Field Cured Specimens

5.12.4.1 The engineer may require strength tests of cylinders cured under field conditions to check adequacy of curing and protection of concrete in the structure.

5.12.4.2 Field cured cylinders shall be cured under field conditions in accordance with "Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C31 / C31M - 09).

5.12.4.3 Field cured test cylinders shall be moulded at the same time and from the same samples as laboratory cured test cylinders.

5.12.4.4 Procedures for protecting and curing concrete shall be improved when the strength of field cured cylinders at the test age designated for determination of f'_c is less than 85 per cent of that of companion laboratory cured cylinders. The 85 per cent limitation shall not apply if field cured strength exceeds f'_c by more than 3.5 N/mm².

5.12.5 Investigation of Low Strength Test Results

- 5.12.5.1 If the result of any strength test (Sec 5.12.2.4) of laboratory cured cylinders falls below the specified value of f'_c by more than 3.5 N/mm² (Sec 5.12.3.3(b)) or if tests of field cured cylinders indicate deficiencies in protection and curing (Sec 5.12.4.4), steps shall be taken to assure that the load carrying capacity of the structure is not jeopardized.
- 5.12.5.2 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 C42 / C42M - 04). In such cases, three cores shall be taken for each strength test more than 3.5 N/mm² below the specified value of f'_c .
- 5.12.5.3 If concrete in the structure is expected to be dry under service conditions, cores shall be air dried for 7 days before test and shall be tested dry. If concrete in the structure is expected to be more than superficially wet under service conditions, cores shall be immersed in water for at least 40 hours and be tested wet.
- 5.12.5.4 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 per cent of f'_c and if no single core is less than 75 per cent of f'_c . Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted.
- 5.12.5.5 If the criteria of Sec 5.12.5.4 above are not met, and if structural adequacy remains in doubt, the responsible authority may order load tests for the questionable portion of the structure, or take other appropriate action.

5.13 PROPERTIES OF CONCRETE

5.13.1 Strength

Strength of concrete shall be based on f'_c determined in accordance with the provisions of Sec 5.12.1.

5.13.2 Modulus of Elasticity

5.13.2.1 Modulus of elasticity E_c for stone aggregate concrete may be taken as $44 \text{ wc}^{1.5} \sqrt{f'_c}$ (N/mm²) for values of wc between 15 and 25 kN/m³ and f'_c in N/mm². For normal density concrete, E_c may be taken as $4700 \sqrt{f'_c}$.

5.13.2.2 Modulus of elasticity E_c for brick aggregate concrete may be taken as $3750 \sqrt{f'_c}$.

5.13.3 Creep

The final (30 year) creep strain in concrete ϵ_{cc} shall be predicted from

$$\epsilon_{cc} = \frac{\text{stress}}{E_t} c_c \quad (5.13.1)$$

where

E_t is the modulus of elasticity of the concrete at the age of loading t ,

c_c is the creep coefficient.

The creep coefficient may be estimated from Fig 6.5.1. In this figure, for uniform sections, the effective section thickness is defined as twice the cross-sectional area divided by the exposed perimeter. If drying is prevented by immersion in water or by sealing, the effective section thickness shall be taken as 600 mm.

It can be assumed that about 40%, 60% and 80% of the final creep develops during the first month, 6 months and 30 months under load respectively, when concrete is exposed to conditions of constant relative humidity.

5.13.4 Shrinkage

An estimate of the drying shrinkage of plain concrete may be obtained from Fig 6.5.2. Recommendations for effective section thickness and relative humidity are given in Sec 5.13.3.

Fig 6.5.2 relates to concrete of normal workability made without water reducing admixtures; such concretes shall have an original water content of about 190 l/m³. Where concrete is known to have a different water content, shrinkage shall be regarded as proportional to water content within the range 150 l/m³ to 230 l/m³.

The shrinkage of plain concrete is primarily dependent on the relative humidity of the air surrounding the concrete, the surface area from which moisture can be lost relative to the volume of concrete and on the mix proportion. It is increased slightly by carbonation and self desiccation and reduced by prolonged curing.

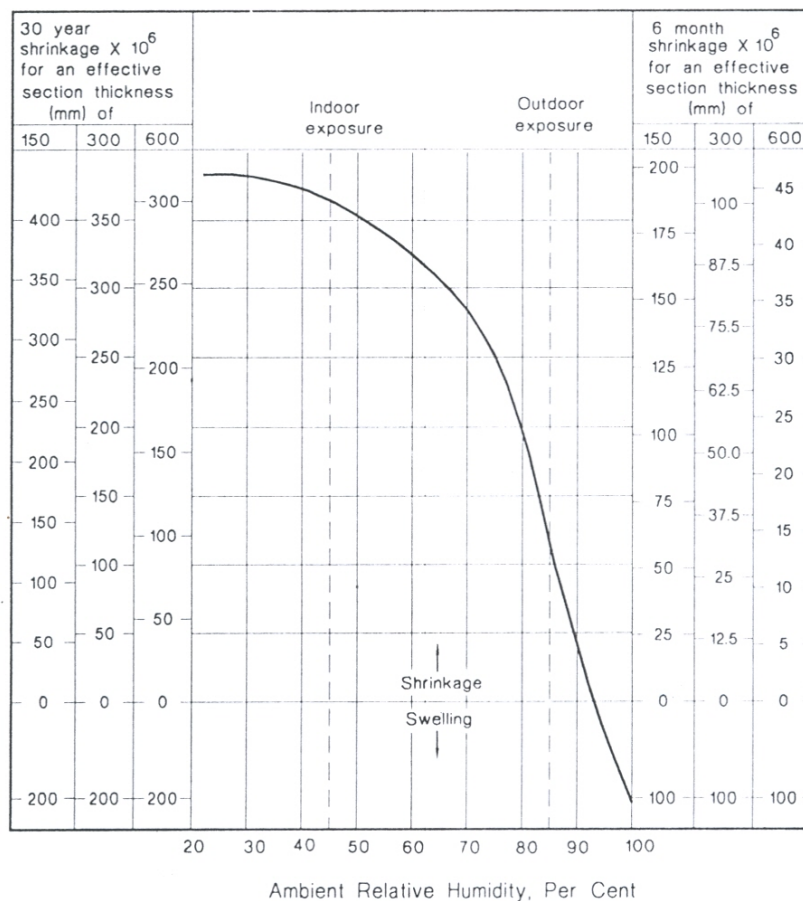


Fig. 6.5.2 Drying Shrinkage of Normal-Weight Concrete

An estimate of the shrinkage of symmetrically reinforced concrete sections may be obtained from :

$$\frac{\epsilon_{sh}}{1 + k\rho} \quad (5.13.2)$$

Where

ϵ_{sh} is the shrinkage of the plain concrete;

ρ is the area of steel relative to that of the concrete;

K is a coefficient, taken as 25 for internal exposure and as 15 for external exposure.

5.13.5 Thermal Strains

Thermal strains shall be calculated from the product of a suitable coefficient of thermal expansion and a temperature change. The temperature change can be determined from the expected service conditions and climatic data. Externally exposed concrete does not respond immediately to air temperature change, and climatic temperature ranges may require adjustment before use in movement calculations.

The coefficient of thermal expansion of concrete is dependent mainly on the expansion coefficients for the aggregate and the cement paste, and the degree of saturation of the concrete. The thermal expansion of aggregate is related to mineralogical composition (See Table 6.5.7)

Table 6.5.7: Thermal Expansion of Rock Group and Related Concrete

Aggregate Type	Typical Coefficient of Expansion ($1 \times 10^{-6}/^{\circ}\text{C}$)	
	Aggregate	Concrete
Flint, quartzite	11	12
Granite, basalt	7	10
Limestone	6	8

Cement paste has a coefficient of thermal expansion that is a function of moisture content, and this affects the concrete expansion as shown in Fig 6.5.3. It may be seen that partially dry concrete has a coefficient of thermal expansion that is approximately $2 \times 10^{-6}/^{\circ}\text{C}$ greater than the coefficient for saturated concrete.

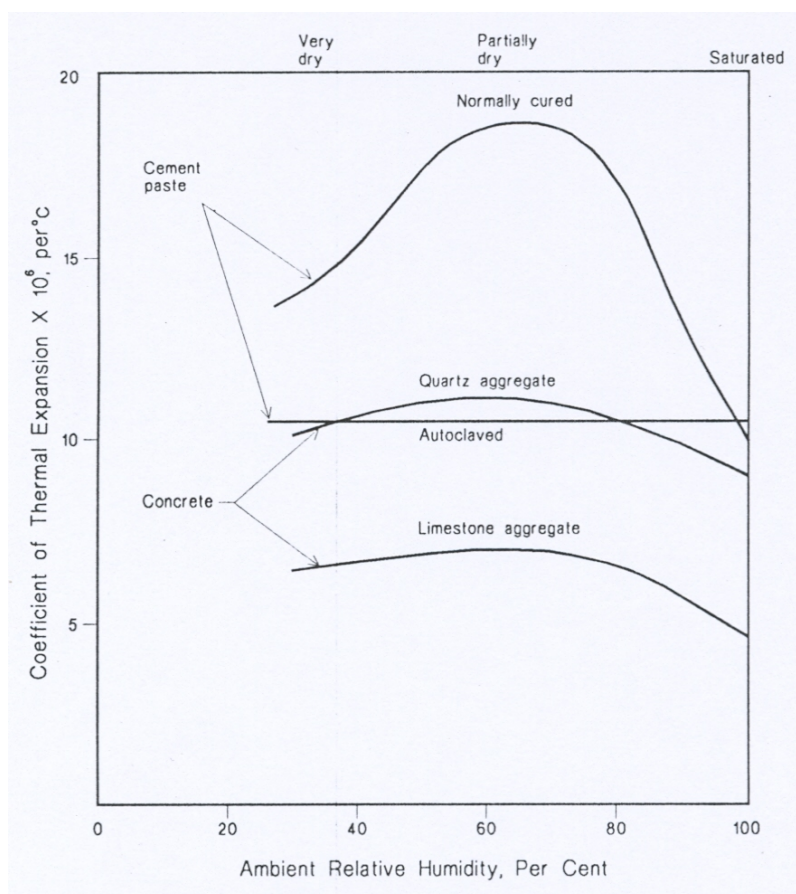


Fig. 6.5.3 Effect of Dryness upon the Coefficient of Thermal Expansion of Hardened Cement and Concrete

5.14 CONCRETING IN ADVERSE WEATHER

5.14.1

Concreting shall be avoided during periods of near freezing weather.

5.14.2

During hot weather, proper attention shall be given to ingredients, production methods, handling, placing, protection, and curing to prevent excessive concrete temperatures or water evaporation that could impair required strength or serviceability of the member or structure.

5.14.3

During rainy weather, proper protection shall be given to ingredients, production methods, handling and placing of concrete. If required in the opinion of the engineer, the concreting operation shall be postponed and newly placed concrete shall be protected from rain after forming proper construction joint for future continuation.

5.15 SURFACE FINISH

5.15.1 Type of Finish

A wide variety of finishes can be produced. Surface cast against forms may be left as cast, e.g. plain or profiled, the initial surface may be removed, e.g. by tooling or sandblasting, or the concrete may be covered, e.g. by paint or tiles; combinations of these techniques may also be adopted, e.g. a ribbed profile with bush hammered ribs. Upper surfaces not cast against forms may be trowelled smooth or profiled, e.g. by tamping; the initial surface may be removed, e.g. by spraying, or it may be covered, e.g. by a screed or plastic floor finish. When selecting the type of finish, consideration shall be given to the ease of producing a finish of the required standard, the viewing distance and the change of appearance with time. In the case of external surfaces, account shall be taken of the weather pattern at the particular location, any impurities in the air and the effect of the shape of the structure upon the flow of water across its surface. Such considerations will often preclude the specification of surfaces of uniform colour as these are very difficult to produce and deteriorate with time, particularly if exposed to the weather.

5.15.2 Quality of Finish

A high quality finish is one that is visually pleasing; it may include colour variations and physical discontinuities but these are likely to be distributed systematically or randomly over the whole surface rather than being concentrated in particular areas. When deciding on the quality of finish to be specified, consideration should be given to the viewing distance and the exposure conditions.

There is no method whereby the quality of finish that will be accepted can unequivocally be defined. To achieve the quality required calls for good communication between experienced personnel conversant with the production of finishes and close collaboration with the site. The quality of finish can be identified in the following very broad terms:

- (a) Class 2 applies to surfaces that are to be exposed to view but where appearance is not critical; such surfaces might be the walls of fire escape stairs or plant rooms and columns and beams of structures that are normally viewed in the shade, e.g. car parks and warehouses;
- (b) Class 1 is appropriate to most surfaces exposed to view including the external walls of industrial, commercial and domestic buildings;
- (c) Special class is appropriate to the highest standards of appearance, such as might be found in prestigious buildings, where it is possible to justify the high cost of their production.
- (d) These broad descriptions may be amplified by written descriptions of the method of finish, by photographs, by samples or by reference to existing structures.

5.15.3 Type of Surface Finish

Smooth off-the-form and board marked finishes are not recommended for external use, but where they are specified for interior use the following types may be quoted for the guidance of both designers and contractor.

Designers should appreciate that it is virtually impossible to achieve dense, flat, smooth, even coloured blemish free concrete surfaces directly from the form work. Some degree of making good is inevitable, even with precast work.

- (a) Type A finish : This finish is obtained by the use of properly designed formwork or moulds of timber, plywood, plastics, concrete or steel. Small blemishes caused by entrapped air or water may be expected, but the surface should be free from voids, honeycombing or other blemishes.
- (b) Type B finish : This finish can only be obtained by the use of high quality concrete and formwork. The concrete shall be thoroughly compacted and all surfaces shall be true, with clean arises. Only very minor surface blemishes shall occur, with no staining or discoloration from the release agent.
- (c) Type C finish : This finish is obtained by first producing a type B finish. The surface is then improved by carefully removing all fins and other projections, thoroughly washing down, and then filling the most noticeable surface blemishes with a cement and fine aggregate paste to match the colour of the original concrete. The release agent should be carefully chosen to ensure that the concrete surface will not be stained or discoloured. After the concrete has been properly cured, the face shall be rubbed down, where necessary, to produce a smooth and even surface.

5.15.4 Production

The quality of a surface depends on the constituents and proportions of the concrete mix, the efficiency of mixing, the handling and compaction of the concrete and its curing. The characteristics of the formwork and the release agent may also be of critical importance. Requirements may be stated for any aspect of production that might contribute towards the achievement of the required type of quality of finish.

5.15.5 Inspection and Making Good

The surface of the concrete shall be inspected for defects and for conformity with the specification and, where appropriate, for comparison with approved sample finishes. Subject to the strength and durability of the concrete being unimpaired, the making good of surface defects may be permitted but the standard of acceptance shall be appropriate to the type and quality of the finish specified and ensure satisfactory performance and durability. On permanently exposed surfaces great care is essential in selecting the materials and the mix proportions to ensure that the final colour of the faced area blends with the parent concrete in the finished structure.

Voids can be filled with fine mortar, preferably incorporating styrene butadiene rubber (SBR) or polyvinyl acetate (PVA), while the concrete is still green or when it has hardened. Fine cracks can be filled by wiping a cement grout, an SBR, PVA or latex emulsion, a cement/SBR or a cement/PVA slurry across them. Fins and other projections shall be rubbed down.

5.15.6 Protection

High quality surface finishes are susceptible to damage during subsequent construction operations and temporary protection may have to be provided in vulnerable areas. Examples of such protective measures include the strapping of laths to arrises and the prevention of rust being carried from exposed starter bars to finished surfaces.

5.16 FORMWORK

5.16.1 Design of Formwork

5.16.1.1 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.

5.16.1.2 Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

5.16.1.3 Forms shall be properly braced or tied together to maintain position and shape.

5.16.1.4 Forms and their supports shall be designed so as not to damage previously placed structure.

5.16.1.5 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.

5.16.1.6 Forms for prestressed concrete members shall be designed and constructed to permit movement of the member without damage during application of prestressing force.

5.16.2 Removal of Forms and Shores

5.16.2.1 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 support safely its weight and loads placed thereon.

5.16.2.2 Sufficient strength shall be demonstrated by structural analysis considering proposed loads, strength of forming and shoring system, and concrete strength data. Structural analysis and concrete strength test data shall be furnished to the engineer when so required.

5.16.2.3 No construction loads exceeding the combinations 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.

5.16.2.4 Forms shall be removed in such a manner as not to impair safety and serviceability of the structure. All concrete to be exposed by form removal shall have sufficient strength not to be damaged thereby.

5.16.2.5 Forms supporting prestressed concrete members shall not be removed until sufficient prestressing has been applied to enable prestressed members to carry their dead load and anticipated construction loads.

5.16.3 Conduits and Pipes Embedded in Concrete

5.16.3.1 Conduits, pipes and sleeves of any materials not harmful to concrete and within the limitations specified herein shall be permitted to be embedded in concrete with the approval of the engineer, provided they are not considered to replace structurally the displaced concrete.

5.16.3.2 Conduits and pipes of aluminium shall not be embedded in structural concrete unless effectively coated or covered to prevent aluminium concrete reaction or electrolytic action between aluminium and steel.

5.16.3.3 Conduits, pipes, and sleeves passing through a slab, wall, or beam shall not impair significantly the strength of the construction.

5.16.3.4 Conduits and pipes, with their fittings, embedded within a column shall not displace more than 4 per cent of the area of cross-section on which strength is calculated or which is required for fire protection.

5.16.3.5 Except when drawings for conduits and pipes are approved by the engineer, conduits and pipes embedded within a slab, wall or beam (other than those merely passing through) shall satisfy the following:

- (a) They shall not be larger in outside dimension than $\frac{1}{3}$ the overall thickness of slab, wall, or beam in which they are embedded.

- (b) They shall not be spaced closer than 3 diameters or widths on centre.
 - (c) They shall not impair significantly the strength of the construction.
- 5.16.3.6 Conduits, pipes and sleeves shall be permitted to be considered as replacing structurally in compression the displaced concrete provided :
- (a) They are not exposed to rusting or other deterioration.
 - (b) They have nominal inside diameter not over 50 mm and are spaced not less than 3 diameters on centres.
- 5.16.3.7 Pipes and fittings shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected.
- 5.16.3.8 No liquid, gas, or vapour, except water not exceeding 30°C nor 0.3 N/mm² pressure, shall be placed in the pipes until the concrete has attained its design strength.
- 5.16.3.9 In solid slabs, piping, unless it is for radiant heating, shall be placed between the top and bottom reinforcements.
- 5.16.3.10 Concrete cover for pipes, conduits, and fittings shall be not less than 40 mm for concrete exposed to earth or weather, nor 20 mm for concrete not exposed to weather or in contact with ground.
- 5.16.3.11 Reinforcement with an area not less than 0.002 times the area of concrete section shall be provided normal to piping.
- 5.16.3.12 Piping and conduit shall be so fabricated and installed that cutting, bending, or displacement of reinforcement will not be required.

5.16.4 Construction Joints

- 5.16.4.1 Surface of concrete construction joints shall be cleaned and laitance removed.
- 5.16.4.2 Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.
- 5.16.4.3 Construction 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 Sec 6.13.3.15(j).
- 5.16.4.4 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.
- 5.16.4.5 Beams, girders, or slabs supported by columns or walls shall not be cast or erected until concrete in the columns or walls is no longer plastic.
- 5.16.4.6 Beams, girders, haunches, drop panels and capitals shall be placed monolithically as part of a slab system unless otherwise shown in the design drawings or specifications.

5.17 SHOTCRETE

5.17.1 General

Shotcrete shall be defined as mortar or concrete pneumatically projected at high velocity onto a surface. Except as specified in this section, shotcrete shall conform to the provisions of this Code regarding plain concrete or reinforced concrete.

5.17.2 Proportions and Materials

Shotcrete proportions shall be such that suitable placement is ensured using the delivery equipment selected, and shall result in finished in place hardened shotcrete meeting the strength requirements of Chapter 6.

5.17.3 Aggregate

Coarse aggregate, if used, shall not exceed 20 mm in size.

5.17.4 Reinforcement

The maximum size of reinforcement shall be 16 mm $\emptyset$ bars unless it can be demonstrated by preconstruction tests that adequate embedment of larger bars can be achieved. When 16 mm $\emptyset$ or smaller bars are used, there shall be a minimum clearance of 60 mm between parallel reinforcing bars. When bars larger than 16 mm $\emptyset$ are permitted, there shall be a minimum clearance between parallel bars equal to six diameters of the bars used. When two curtains of steel are provided, the curtain nearest the nozzle shall have a spacing equal to 12 bar diameters and the remaining curtain shall have a minimum spacing of 6 bar diameters.

Lap splices in reinforcing bars shall be by the noncontact lap splice method with at least 50 mm clearance between bars. The engineer may permit the use of contact lap splices when necessary for the support of the reinforcement, provided it can be demonstrated by means of preconstruction testing that adequate embedment of the bars at the splice can be achieved and provided further that the splices are placed so that the plane containing the centres of the two spliced bars is perpendicular to the surface of the shotcrete work. Shotcrete shall not be applied to spirally tied columns.

5.17.5 Preconstruction Tests

When required by the engineer a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project. The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and the most congested area specified in the structural design. It shall be shot at the same angle, from a similar distance, using the same nozzleman and with the same concrete mix design that will be used on the project.

5.17.6 Rebound

Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be reused as aggregate.

5.17.7 Joints

Except where permitted, unfinished work shall not be allowed to stand for more than 30 minutes unless all edges are sloped thin. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

5.17.8 Damage

An in-place shotcrete which exhibits sags or sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced.

5.17.9 Curing

During the curing periods, shotcrete shall be maintained above 5oC and in moist condition. In initial curing, shotcrete shall be kept continuously moist for 24 hours after placement is complete. Final curing shall continue for seven days after shotcreting, for three days if high early strength cement is used, or until the specified strength is obtained. Final curing shall consist of a fog spray or an approved moisture retaining cover or membrane. In sections of a depth in excess of 300 mm, final curing shall be the same as that for initial curing.

5.17.10 Strength Test

Strength test for shotcrete shall be made by an approved agency on specimens which are representative of the work and which have been water soaked for at least 24 hours prior to testing. When the maximum size aggregate is larger than 10 mm, specimens shall consist of not less than three 75 mm diameter cores or 75 mm cubes. When the maximum size aggregate is 10 mm or smaller, specimens shall consist of not less than three 50 mm diameter cores or 50 mm cubes. Specimens shall be taken in accordance with one of the following:

- (a) From the work: taken at least one from each shift but not less than one for each 20 m³ of shotcrete;
- (b) From test panels: taken not less than once each shift nor less than one for each 20 m³ of shotcrete placed. When the maximum size aggregate is larger than 10 mm, the test panels shall have a minimum dimension of 450 mm by 450 mm. When the maximum size aggregate is 10 mm or smaller, the test panels shall have a minimum dimension of 300 mm by 300 mm. Panels shall be gunned in the same position as the work, during the course of the work and by the same nozzlemen doing the work. The condition under which the panels are cured shall be the same as the work.

The average strength of three cores from a single panel shall be equal to or exceed $0.85 f'_c$ with no single core less than $0.75 f'_c$. The average strength of three cubes taken from a single panel must equal or exceed f'_c with no individual cube less than f'_c . To check testing accuracy, locations represented by erratic core strengths may be retested.

5.17.11 Inspections

5.17.11.1 Inspection During Placement

When shotcrete is used for columns and beams, a special inspector is required. The special inspector shall provide continuous inspection to the placement of the reinforcement and shotcreting and shall submit a statement indicating compliance with the plans and specifications.

5.17.11.2 Visual Examination for Structural Soundness of In-place Shotcrete

Completed shotcrete work shall be checked visually for reinforcing bar embedment, voids, rock pocket, sand streaks and similar deficiencies by examining a minimum of three 75 mm cores taken from three areas chosen by the engineer which represent the worst congestion of reinforcing bars occurring in the project. Extra reinforcing bars may be added to noncongested areas and cores may be taken from these areas. The cores shall be examined by the special inspector and a report submitted to the engineer prior to final approval of the shotcrete.

5.17.12 Equipment

The equipment used in construction testing shall be the same equipment used in the work requiring such testing unless substitute equipment is approved by the engineer.

Strength Design of Reinforced Concrete Structures

6.1 Analysis and Design - General Considerations

6.1.1 Convention and Notation

Unless otherwise explicitly stated, the following units shall be implicit for the corresponding quantities in the design and other expressions provided in this chapter:

Lengths	m m
Areas	mm ²
Second moments of area	mm ⁴
Force (axial, shear)	N
Moment, torsion	N m m
Stress, strength	MPa, N/mm ²

6.1.1.1 Notation

a	= Depth of equivalent rectangular stress block as defined in 6.3.2.7.1, mm,
a_v	= Shear span, equal to distance from center of concentrated load to either: (a) face of support for continuous or cantilevered members, or (b) center of support for simply supported members, mm, Sec 6.4, Appendix A
A_b	= Area of an individual bar or wire, mm ² , Sec 6.3, Sec 8.2
A_{brg}	= Net bearing area of the head of stud, anchor bolt, or headed deformed bar, mm ² , Sec 8.2, Appendix D
A_c	= Cross-sectional area of concrete section resisting shear transfer, mm ² , Sec 6.4, Sec 8.3
A_{ch}	= Cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, mm ² , Sec 6.3, Sec 8.3
A_{cp}	= Area enclosed by outside perimeter of concrete cross section, mm ² , see 6.4.4.1, Sec 6.4, 8.3.8.3
A_{cs}	= Cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, mm ² , Appendix A
A_{cv}	= Gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm ² , Sec 8.3
A_{cw}	= Area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear, mm ² , Sec 8.3
A_f	= Area of reinforcement in bracket or corbel resisting factored moment, mm ² , see 6.4.7, Sec 6.4
A_g	= Gross area of concrete section, mm ² For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s), see 6.4.4.1, Secs 6.2 to 6.4, 6.6, 6.7, 6.10, 8.3,
A_h	= Total area of shear reinforcement parallel to primary tension reinforcement in a corbel or bracket, mm ² , see 6.4.7, Sec 6.4
A_j	= Effective cross-sectional area within a joint in a plane parallel to plane of reinforcement generating shear in the joint, mm ² , see Sec 8.3
A_l	= Total area of longitudinal reinforcement to resist torsion, mm ² , Sec 6.4, 8.3
$A_{l,min}$	= Minimum area of longitudinal reinforcement to resist torsion, mm ² , see 6.4.4.5.3, Sec 6.4

A_n	= Area of reinforcement in bracket or corbel resisting tensile force N_{uc} , mm ² , see 6.4.7, Sec 6.4
A_{nz}	= Area of a face of a nodal zone or a section through a nodal zone, mm ² , Appendix A
A_{Nc}	= Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, mm ² , see D.5.2.1, Appendix D
A_{Nco}	= Projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, mm ² , see D.5.2.1, Appendix D
A_o	= Gross area enclosed by shear flow path, mm ² , Sec 6.4
A_{oh}	= Area enclosed by centerline of the outermost closed transverse torsional reinforcement, mm ² , Sec 6.4
A_s	= Area of nonprestressed longitudinal tension reinforcement, mm ² , Sec 6.3, 6.4, 6.6, 6.8,
A_{s1}	= Area of tension reinforcement corresponding to moment of resistance M_{n1} , see 6.3.15.1(b)
A_{s2}	= Area of additional tension steel, see 6.3.15.1(b)
A'_s	= Area of compression reinforcement, mm ² , Appendix A
A_{sc}	= Area of primary tension reinforcement in a corbel or bracket, mm ² , see 6.4.7.3.5, Sec 6.4
$A_{se,N}$	= Effective cross-sectional area of anchor in tension, mm ² , Appendix D
$A_{se,V}$	= Effective cross-sectional area of anchor in shear, mm ² , Appendix D
A_{sf}	= Area of reinforcement required to balance the longitudinal compressive force in the overhanging portion of the flange of a T-beam, see 6.3.15.2(b)
A_{sh}	= Total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension h_c , mm ² , Sec 8.3
A_{si}	= Total area of surface reinforcement at spacing s_i in the i -th layer crossing a strut, with reinforcement at an angle α_i to the axis of the strut, mm ² , Appendix A
$A_{s,min}$	= Minimum area of flexural reinforcement, mm ² , see 6.3.5, Sec 6.3
A_{st}	= Total area of nonprestressed longitudinal reinforcement (bars or steel shapes), mm ² , Sec 6.3, 8.3
A_{sx}	= Area of structural steel shape, pipe, or tubing in a composite section, mm ² , Sec 6.3
A_t	= Area of one leg of a closed stirrup resisting torsion within spacing s , mm ² , Sec 6.4
A_{tp}	= Area of prestressing steel in a tie, mm ² , Appendix A
A_{tr}	= Total cross-sectional area of all transverse reinforcement within spacing s that crosses the potential plane of splitting through the reinforcement being developed, mm ² , Sec 8.2
A_{ts}	= Area of nonprestressed reinforcement in a tie, mm ² , Appendix A
A_v	= Area of shear reinforcement spacing s , mm ² , Sec 6.4, 6.12
A_{Vc}	= Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, mm ² , see D.6.2.1, Appendix D
A_{Vco}	= Projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, mm ² , see D.6.2.1, Appendix D
A_{vd}	= Total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, mm ² , Sec 8.3
A_{vf}	= Area of shear-friction reinforcement, mm ² , Sec 6.4, 8.3
A_{vh}	= Area of shear reinforcement parallel to flexural tension reinforcement within spacing s_2 , mm ² , Sec 6.4
$A_{v,min}$	= Minimum area of shear reinforcement within spacing s , mm ² , see 6.4.3.5.1 and 6.4.3.5.3, Sec 6.4
A_1	= Loaded area, mm ² , Sec 6.3
A_2	= Area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal, mm ² , Sec 6.3
b	= Width of compression face of member, mm, Sec 6.3
b_o	= Perimeter of critical section for shear in slabs and footings, mm, see 6.4.10.1.2, Sec 6.4
b_s	= Width of strut, mm, Appendix A
b_t	= Width of that part of cross section containing the closed stirrups resisting torsion, mm, Sec

	6.4
b_v	= Width of cross section at contact surface being investigated for horizontal shear, mm, Sec 6.12
b_w	= Web width, or diameter of circular section, mm, Sec 6.3, 6.4, 8.2, 8.3
b_1	= Dimension of the critical section b_o measured in the direction of the span for which moments are determined, mm, Sec 6.5
b_2	= Dimension of the critical section b_o measured in the direction perpendicular to b_1 , mm, Sec 6.5
c	= Distance from extreme compression fiber to neutral axis, mm, Sec 6.2, 6.3, 6.6, 8.3
c_{ac}	= Critical edge distance required to develop the basic concrete breakout strength of a post-installed anchor in uncracked concrete without supplementary reinforcement to control splitting, mm, see D.8.6, Appendix D
$c_{a,max}$	= Maximum distance from center of an anchor shaft to the edge of concrete, mm, Appendix D
$c_{a,min}$	= Minimum distance from center of an anchor shaft to the edge of concrete, mm, Appendix D
c_{a1}	= Distance from the center of an anchor shaft to the edge of concrete in one direction, mm. If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance, appendix d
c_{a2}	= Distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1} , mm, Appendix D
c_b	= Smaller of: (a) the distance from center of a bar or wire to nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires being developed, mm, Sec 8.2
c_c	= Clear cover of reinforcement, mm, see 6.3.6.4, Sec 6.3
c_t	= Distance from the interior face of the column to the slab edge measured parallel to c_1 , but not exceeding c_1 , mm, Sec 8.3
c_1	= Dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm, Sec 6.4, 6.5, 8.3
c_2	= Dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1 , mm, Sec 6.5
C	= Cross-sectional constant to define torsional properties of slab and beam, see 6.5.6.4.2, Sec 6.5
C_m	= Factor relating actual moment diagram to an equivalent uniform moment diagram, Sec 6.3
d	= Distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm, Sec 6.2-6.4, 6.6, 6.12, 8.1-8.3,
d'	= Distance from extreme compression fiber to centroid of longitudinal compression reinforcement, mm, Sec 6.2
d_a	= Outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, mm, see D.8.4, Appendix D
d'_a	= Value substituted for d_a when an oversized anchor is used, mm, see D.8.4, Appendix D
d_b	= Nominal diameter of bar, wire, or prestressing strand, mm, Sec 8.1-8.3
d_p	= Distance from extreme compression fiber to centroid of prestressing steel, mm, Sec 6.4
d_{pile}	= Diameter of pile at footing base, mm, Sec 6.8
d_t	= Distance from extreme compression fiber to centroid of extreme layer of longitudinal tension steel, mm, Sec 6.2, 6.3
D	= Dead loads, or related internal moments and forces, Sec 6.1, 6.2, 6.11, 8.3
e_h	= Distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt, mm, Appendix D
e'_N	= Distance between resultant tension load on a group of anchors loaded in tension and the Centroid of the group of anchors loaded in tension, mm; e'_n is always positive, appendix d
e'_V	= Distance between resultant shear load on a group of anchors loaded in shear in the same Direction, and the centroid of the group of anchors loaded in shear in the same direction, mm; e'_v is always positive, appendix d
E	= Load effects of earthquake, or related internal moments and forces, Sec 6.2, 8.3
E_c	= Modulus of elasticity of concrete, mpa, see 6.1.7.1, Sec 6.1-6.3, 6.6, 6.9

E_{cb}	= Modulus of elasticity of beam concrete, mpa, Sec 6.5
E_{cs}	= Modulus of elasticity of slab concrete, mpa, Sec 6.5
EI	= Flexural stiffness of compression member, N·mm ² , see 6.3.10.6, Sec 6.3
E_p	= Modulus of elasticity of prestressing steel, mpa, see 6.1.7.3, Sec 6.1
E_s	= Modulus of elasticity of reinforcement and structural steel, mpa, see 6.1.7.2, Sec 6.1, 6.3, 6.6
f'_c	= Specified compressive strength of concrete, mpa, Sec 6.1-6.4, 6.6, 6.9, 8.2, 8.3, Appendixes A, D
$\sqrt{f'_c}$	= Square root of specified compressive strength of concrete, mpa, Sec 6.1, 6.2, 6.4, 6.9, 8.2, 8.3, Appendix D
f_{ce}	= Effective compressive strength of the concrete in a strut or a nodal zone, mpa, Sec 6.8, Appendix A
f_{ct}	= Average splitting tensile strength of lightweight concrete, mpa, See 6.1.8.1 Sec 6.1, 6.4, 8.2.3.4 (d), Sec 8.2
f_d	= Stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, mpa, Sec 6.4
f_{pc}	= Compressive stress in concrete (after allowance for all prestress 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 the 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 prestress and moments resisted by precast member acting alone), Sec 6.4
f_{pe}	= Compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, mpa, Sec 6.4
f_{ps}	= Stress in prestressing steel at nominal flexural strength, mpa, Sec 8.2
f_{pu}	= Specified tensile strength of prestressing steel, mpa, Sec 6.4
f_r	= Modulus of rupture of concrete, mpa, see 6.2.5.2.3, Sec 6.2, 6.6
f_s	= Calculated tensile stress in reinforcement at service loads, mpa, Sec 6.3
f'_s	= Stress in compression reinforcement under factored loads, mpa, Appendix A
f_{se}	= Effective stress in prestressing steel (after allowance for all prestress losses), mpa, Sec 8.2, Appendix A
f_{uta}	= Specified tensile strength of anchor steel, mpa, Appendix D
f_y	= Specified yield strength of reinforcement, mpa, Sec 6.2-6.4, 6.6, 6.9, 6.12, 8.1-8.3, Appendix A
f_{ya}	= Specified yield strength of anchor steel, mpa, Appendix D
f_{yt}	= Specified yield strength f_y of transverse reinforcement, mpa, Sec 6.3, 6.4, 8.2-8.3
F	= Loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces, Sec 6.2
F_n	= Nominal strength of a strut, tie, or nodal zone, N, Appendix A
F_{nn}	= Nominal strength at face of a nodal zone, N, Appendix A
F_{ns}	= Nominal strength of a strut, N, Appendix A
F_{nt}	= Nominal strength of a tie, N, Appendix A
F_u	= Factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model, N, Appendix A
h	= Overall thickness or height of member, mm, Sec 6.2-6.4, 6.6, 6.11, 6.12, 8.2, 8.3, Appendix A
h_a	= Thickness of member in which an anchor is located, measured parallel to anchor axis, mm, Appendix D
h_c	= Cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh} , mm, Sec 8.3
h_{ef}	= Effective embedment depth of anchor, mm, see D.8.5, Appendix D
h_f	= Thickness of overhanging portion of the flange of a T-beam, see 6.3.15.2(b)
h_v	= Depth of shearhead cross section, mm, Sec 6.4
h_w	= Height of entire wall from base to top or height of the segment of wall considered, mm, Sec 6.4, 8.3
h_x	= Maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of the column, mm, Sec 8.3

H	= Loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces, Sec 6.2
I	= Moment of inertia of section about centroidal axis, mm^4 , Sec 6.3, 6.4
I_b	= Moment of inertia of gross section of beam about centroidal axis, mm^4 , see 6.5.6.1.6, Sec 6.5
I_{cr}	= Moment of inertia of cracked section transformed to concrete, mm^4 , Sec 6.2
I_e	= Effective moment of inertia for computation of deflection, mm^4 , see 6.2.5.2.3, Sec 6.2
I_g	= Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4 , Sec 6.2, 6.3, 6.6
I_s	= Moment of inertia of gross section of slab about centroidal axis defined for calculating α_f and β_t , mm^4 , Sec 6.5
I_{se}	= Moment of inertia of reinforcement about centroidal axis of member cross section, mm^4 , Sec 6.3
I_{sx}	= Moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, mm^4 , Sec 6.3
k	= Effective length factor for compression members, Sec 6.3, 6.6
k_c	= Coefficient for basic concrete breakout strength in tension, Appendix D
k_{cp}	= Coefficient for pryout strength, Appendix D
K_{tr}	= Transverse reinforcement index, see 8.2.3.3, Sec 8.2
l	= Span length of beam or one-way slab; clear projection of cantilever, mm, Sec 6.2
l_a	= Additional embedment length beyond centerline of support or point of inflection, mm, Sec 8.2
l_c	= Length of compression member in a frame, measured center-to-center of the joints in the frame, mm, Sec 6.3, 6.6
l_d	= Development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand, mm, Sec 6.9, 8.1-8.3
l_{dc}	= Development length in compression of deformed bars and deformed wire, mm, Sec 8.2
l_{dh}	= Development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), mm, see Sec. 8.2 and 8.3, Sec 8.2, 8.3
l_{dt}	= Development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head, mm, Sec 8.2
l_e	= Load bearing length of anchor for shear, mm, see D.6.2.2, Appendix D
l_n	= Length of clear span measured face-to-face of supports, mm, Sec 6.1-6.5, 6.10, 8.2.9.3, Sec 8.2, 8.3
l_o	= Length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, mm, Sec 8.3
l_t	= Span of member under load test, taken as the shorter span for two-way slab systems, mm. Span is the smaller of: (a) distance between centers of supports, and (b) clear distance between supports plus thickness h of member. Span for a cantilever shall be taken as twice the distance from face of support to cantilever end, Sec 6.11
l_u	= Unsupported length of compression member, mm, see 6.3.10.1.1, Sec 6.3
l_v	= Length of shearhead arm from centroid of concentrated load or reaction, mm, Sec 6.4
l_w	= Length of entire wall or length of segment of wall considered in direction of shear force, mm, Sec 6.4, 6.6, 8.3
l_1	= Length of span in direction that moments are being determined, measured center-to-center of supports, mm, Sec 6.5
l_2	= Length of span in direction perpendicular to l_1 , measured center-to-center of supports, mm, see 6.5.6.2.3 and 6.5.6.2.4, Sec 6.5
L	= Live loads, or related internal moments and forces, Sec 6.1, 6.2, 6.11, 8.3
L_r	= Roof live load, or related internal moments and forces, Sec 6.2
M_a	= Maximum moment in member due to service loads at stage deflection is computed, N·mm, Sec 6.2, 6.6
M_c	= Factored moment amplified for the effects of member curvature used for design of compression member, N·mm, see 6.3.10.6, Sec 6.3
M_{cr}	= Cracking moment, N·mm, see 6.2.5.2.3, Sec 6.2, 6.6

- M_{cre} = Moment causing flexural cracking at section due to externally applied loads, N·mm, Sec 6.4
- M_m = Factored moment modified to account for effect of axial compression, N·mm, see 6.4.2.2.2, Sec 6.4
- M_{max} = Maximum factored moment at section due to externally applied loads, N·mm, Sec 6.4
- M_n = Nominal flexural strength at section, N·mm, Sec 6.4, 6.6, 8.2, 8.3
- M_{n1} = Nominal flexural strength at section without compression steel, see 6.3.15.1(b), and moment of resistance developed by compression in the overhanging portion of the T-flange, see 6.3.15.2(b)
- M_{n2} = Additional nominal flexural strength at section due to added compression steel A'_s and additional tension steel A_{s2} , see 6.3.15.1(b), and moment of resistance developed by the web of a T-beam, see 6.3.15.2(b)
- M_{nc} = Nominal flexural strength of column framing into joint, calculated for factored axial force, consistent with the direction of lateral forces considered, resulting in lowest flexural strength, N·mm, Sec 8.3
- M_o = Total factored static moment, N·mm, Sec 6.5
- M_p = Required plastic moment strength of shearhead cross section, N·mm, Sec 6.4
- M_{pr} = Probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile stress in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor, ϕ , of 1.0, N·mm, Sec 8.3
- M_s = Factored moment due to loads causing appreciable sway, N·mm, Sec 6.3
- M_{slab} = Portion of slab factored moment balanced by support moment, N·mm, Sec 8.3
- M_u = Factored moment at section, N·mm, Sec 6.3-6.6, 8.3
- M_{ua} = Moment at midheight of wall due to factored lateral and eccentric vertical loads, not including P_Δ effects, N·mm, Sec 6.6
- M_v = Moment resistance contributed by shearhead reinforcement, N·mm, Sec 6.4
- M_1 = Smaller factored end moment on a compression member, to be taken as positive if member is bent in single curvature, and negative if bent in double curvature, N·mm, Sec 6.3
- M_{1ns} = Factored end moment on a compression member at the end at which M_1 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N·mm, Sec 6.3
- $M_{2,min}$ = Minimum value of M_2 , N·mm, Sec 6.3
- M_{2ns} = Factored end moment on compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, calculated using a first-order elastic frame analysis, N·mm, Sec 6.3
- M_{2s} = Factored end moment on compression member at the end at which M_2 acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N·mm, Sec 6.3
- n = Number of items, such as strength tests, bars, wires, monostrand anchorage devices, anchors, or shearhead arms, Sec 6.4, 8.2, Appendix D
- N_b = Basic concrete breakout strength in tension of a single anchor in cracked concrete, N, see D.5.2.2, Appendix D
- N_{cb} = Nominal concrete breakout strength in tension of a single anchor, N, see D.5.2.1, Appendix D
- N_{cbg} = Nominal concrete breakout strength in tension of a group of anchors, N, see D.5.2.1, Appendix D
- N_n = Nominal strength in tension, N, Appendix D
- N_p = Pullout strength in tension of a single anchor in cracked concrete, N, see D.5.3.4 and D.5.3.5, Appendix D
- N_{pn} = Nominal pullout strength in tension of a single anchor, N, see D.5.3.1, Appendix D
- N_{sa} = Nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, N, see D.5.1.1 and D.5.1.2, Appendix D
- N_{sb} = Side-face blowout strength of a single anchor, N, Appendix D
- N_{sbg} = Side-face blowout strength of a group of anchors, N, Appendix D
- N_u = Factored axial force normal to cross section occurring simultaneously with V_u or T_u ; to be taken as positive for compression and negative for tension, N, Sec 6.4
- N_{ua} = Factored tensile force applied to anchor or group of anchors, N, Appendix D
- N_{uc} = Factored horizontal tensile force applied at top of bracket or corbel acting simultaneously with

	V_u , to be taken as positive for tension, N, Sec 6.4
p_{cp}	= Outside perimeter of concrete cross section, mm, see 6.4.4.1, Sec 6.4
p_h	= Perimeter of centerline of outermost closed transverse torsional reinforcement, mm, Sec 6.4
P_b	= Nominal axial strength at balanced strain conditions, N, see 6.3.3.2, Sec 6.2, 6.3
P_c	= Critical buckling load, N, see 6.3.10.6, Sec 6.3
P_n	= Nominal axial strength of cross section, N, Sec 6.2, 6.3, 6.6
$P_{n,max}$	= Maximum allowable value of P_n , N, see 6.3.3.6, Sec 6.3
P_o	= Nominal axial strength at zero eccentricity, N, Sec 6.3
P_s	= Unfactored axial load at the design (midheight) section including effects of selfweight, N, Sec 6.6
P_u	= Factored axial force; to be taken as positive for compression and negative for tension, N, Sec 6.3, 6.6, 8.3
q_{Du}	= Factored dead load per unit area, Sec 6.5
q_{Lu}	= Factored live load per unit area, Sec 6.5
q_u	= Factored load per unit area, Sec 6.5
Q	= Stability index for a story, see 6.3.10.5.2, Sec 6.3
r	= Radius of gyration of cross section of a compression member, mm, Sec 6.3
R	= Rain load, or related internal moments and forces, Sec 6.2
s	= Center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, prestressing tendons, wires, or anchors, mm, Sec 6.3, 6.4, 6.9, 6.11, 6.12, 8.2, 8.3, Appendix D
s_i	= Center-to-center spacing of reinforcement in the i -th layer adjacent to the surface of the member, mm, Appendix A
s_o	= Center-to-center spacing of transverse reinforcement within the length l_o , mm, Sec 8.3
s_s	= Sample standard deviation, mpa, Appendix D
s_2	= Center-to-center spacing of longitudinal shear or torsion reinforcement, mm, Sec 6.4
S	= Snow load, or related internal moments and forces, Sec 6.2, 8.3
S_e	= Moment, shear, or axial force at connection corresponding to development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects, Sec 8.3
S_n	= Nominal flexural, shear, or axial strength of connection, Sec 8.3
S_y	= Yield strength of connection, based on f_y , for moment, shear, or axial force, Sec 8.3
t	= Wall thickness of hollow section, mm, Sec 6.4
T	= Cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, Sec 6.2
T_n	= Nominal torsional moment strength, N·mm, Sec 6.4
T_u	= Factored torsional moment at section, N·mm, Sec 6.4
U	= Required strength to resist factored loads or related internal moments and forces, Sec 6.2
v_n	= Nominal shear stress, mpa, see 6.4.10.6.2, Sec 6.4, 8.3
V_b	= Basic concrete breakout strength in shear of a single anchor in cracked concrete, N, see D.6.2.2 and D.6.2.3, Appendix D
V_c	= Nominal shear strength provided by concrete, N, Sec 6.1, 6.4, 6.5, 8.3
V_{cb}	= Nominal concrete breakout strength in shear of a single anchor, N, see D.6.2.1, Appendix D
V_{cbg}	= Nominal concrete breakout strength in shear of a group of anchors, N, see D.6.2.1, Appendix D
V_{ci}	= Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, N, Sec 6.4
V_{cp}	= Nominal concrete pryout strength of a single anchor, N, see D.6.3.1, Appendix D
V_{cpg}	= Nominal concrete pryout strength of a group of anchors, N, see D.6.3.1, Appendix D
V_{cw}	= Nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web, N, Sec 6.4
V_d	= Shear force at section due to unfactored dead load, N, Sec 6.4
V_e	= Design shear force corresponding to the development of the probable moment strength of the member, N, Sec 8.3
V_i	= Factored shear force at section due to externally applied loads occurring simultaneously with M_{max} , N, Sec 8.3
V_n	= Nominal shear strength, N, Sec 6.1, 6.3, 6.4, 8.3, Appendix D
V_{nh}	= Nominal horizontal shear strength, N, Sec 6.12

V_p	= Vertical component of effective prestress force at section, N, Sec 6.4
V_s	= Nominal shear strength provided by shear reinforcement, N, Sec 6.4
V_{sa}	= Nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, N, see D.6.1.1 and D.6.1.2, Appendix D
V_u	= Factored shear force at section, N, Sec 6.4, 6.5, 6.12, 8.2, 8.3
V_{ua}	= Factored shear force applied to a single anchor or group of anchors, N, Appendix D
V_{ug}	= Factored shear force on the slab critical section for two-way action due to gravity loads, N, see Sec. 8.3
V_{us}	= Factored horizontal shear in a story, N, Sec 6.3
wl_c	= Density (unit weight) of normalweight concrete or equilibrium density of lightweight concrete, kg/m ³ , Sec 6.1, 6.2
w_u	= Factored load per unit length of beam or oneway slab, Sec 6.1
W	= Wind load, or related internal moments and forces, Sec 6.2
x	= Shorter overall dimension of rectangular part of cross section, mm, Sec 6.5
y	= Longer overall dimension of rectangular part of cross section, mm, Sec 6.5
y_t	= Distance from centroidal axis of gross section, neglecting reinforcement, to tension face, mm, Sec 6.2, 6.4
α	= Angle defining the orientation of reinforcement, Sec 6.4, 8.3, Appendix A
α_c	= Coefficient defining the relative contribution of concrete strength to nominal wall shear strength, Sec 8.3
α_f	= 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, see 6.5.6.1.6, Sec 6.2, 6.5
α_{fm}	= Average value of α_f for all beams on edges of a panel, Sec 6.2
α_{f1}	= A_f in direction of l_1 , Sec 6.5
α_{f2}	= A_f in direction of l_2 , Sec 6.5
α_i	= Angle between the axis of a strut and the bars in the i -th layer of reinforcement crossing that strut, Appendix A
α_s	= Constant used to compute V_c in slabs and footings, Sec 6.4
α_v	= Ratio of flexural stiffness of shearhead arm to that of the surrounding composite slab section, see 6.4.10.4.5, Sec 6.4
β	= Ratio of long to short dimensions: clear spans for two-way slabs, see 6.2.5.3.3; sides of column, concentrated load or reaction area, see 6.4.10.2.1; or sides of a footing, see 6.8.4.4.2, Sec 6.2, 6.4, 6.8
β_b	= Ratio of area of reinforcement cut off to total area of tension reinforcement at section, Sec 8.2
β_{dns}	= Ratio used to account for reduction of stiffness of columns due to sustained axial loads, see 6.3.10.6.2, Sec 6.3
β_{ds}	= Ratio used to account for reduction of stiffness of columns due to sustained lateral loads, see 6.3.10.4.2, Sec 6.3
β_n	= Factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone, Appendix A
β_p	= Factor used to compute V_c in prestressed slabs, Sec 6.4
β_s	= Factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut, Appendix A
β_t	= Ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports, see 6.5.6.4.2, Sec 6.5
β_1	= Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, see 6.3.2.7.3, Sec 6.3
γ_f	= Factor used to determine the unbalanced moment transferred by flexure at slab-column connections, see 6.5.5.3.2, Sec 6.4, 6.5, 8.3
γ_s	= Factor used to determine the portion of reinforcement located in center band of footing, see 6.8.4.4.2, Sec 6.8
γ_v	= Factor used to determine the unbalanced moment transferred by eccentricity of shear at slab-column connections, see 6.4.10.7.1, Sec 6.4

δ	= Moment magnification factor to reflect effects of member curvature between ends of compression member, Sec 6.3
δ_s	= Moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads, Sec 6.3
δ_u	= Design displacement, mm, Sec 8.3
Δ_{cr}	= Computed, out-of-plane deflection at midheight of wall corresponding to cracking moment, M_{cr} , mm, Sec 6.6
Δ_{fp}	= Increase in stress in prestressing steel due to factored loads, mpa, Appendix A
Δ_n	= Computed, out-of-plane deflection at midheight of wall corresponding to nominal flexural strength, M_n , mm, Sec 6.6
Δ_o	= Relative lateral deflection between the top and bottom of a story due to lateral forces computed using a first-order elastic frame analysis and stiffness values satisfying 6.3.10.5.2, mm, Sec 6.3
Δ_r	= Difference between initial and final (after load removal) deflections for load test or repeat load test, mm, Sec 6.11
Δ_s	= Computed, out-of-plane deflection at midheight of wall due to service loads, mm, Sec 6.6
Δ_u	= Computed deflection at midheight of wall due to factored loads, mm, Sec 6.6
Δ_1	= Measured maximum deflection during first load test, mm, see 6.11.5.2, Sec 6.11
Δ_2	= Maximum deflection measured during second load test relative to the position of the structure at the beginning of second load test, mm, see 6.11.5.2, Sec 6.11
ε_t	= Net tensile strain in extreme layer of longitudinal tension steel at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature, Sec 6.1-6.3
θ	= Angle between axis of strut, compression diagonal, or compression field and the tension chord of the member, Sec 6.4, Appendix A
λ	= Modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normalweight concrete of the same compressive strength, see 6.1.8.1, 6.4.5.4.3, 8.2.3.4(d), 8.2.6.2, 8.2.10.2(b), Sec 6.2, 6.4, 6.9, 8.2, 8.3 and Appendixes A, D
λ_Δ	= Multiplier for additional deflection due to long-term effects, see 6.2.5.2.5, Sec 6.2
μ	= Coefficient of friction, see 6.4.5.4.3, Sec 6.4, 8.3
ξ	= Time-dependent factor for sustained load, see 6.2.5.2.5, Sec 6.2
ρ	= Ratio of A_s to bd , Sec 6.4, 6.5, 8.3
ρ'	= Ratio of A'_s to bd , see 6.3.15.1(b), Sec 6.2
ρ_b	= Ratio of A_s to bd producing balanced strain conditions, see 6.3.3.2, Sec 6.3, 6.5, 6.6
ρ_f	= Ratio of A_{sf} to $b_w d$, see 6.3.15.2(b)
ρ_l	= Ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement, Sec 6.4, 6.6, 8.3
ρ_{max}	= Maximum reinforcement ratio allowed for beams corresponding to $\varepsilon_t = 0.004$, see 6.3.15.1(a)
ρ_s	= Ratio of volume of spiral reinforcement to total volume of core confined by the spiral (measured out-to-out of spirals), Sec 6.3, 8.3
ρ_t	= Ratio of area distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement, Sec 6.4, 6.6, 8.3
ρ_v	= Ratio of tie reinforcement area to area of contact surface, see 6.12.5.3.3, Sec 6.12
ρ_w	= Ratio of A_s to $b_w d$, see 6.3.15.2(b), Sec 6.4
φ	= Strength reduction factor, see 6.2.3, Sec 6.1-6.6, 6.9, 6.11, 6.12, 8.3, Appendixes A & D
$\psi_{c,N}$	= Factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete, see D.5.2.6, Appendix D
$\psi_{c,P}$	= Factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete, see D.5.3.6, Appendix D
$\psi_{c,V}$	= Factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement, see D.6.2.7 for anchors in shear, Appendix D
$\psi_{cp,N}$	= Factor used to modify tensile strength of postinstalled anchors intended for use in uncracked concrete without supplementary reinforcement, see D.5.2.7, Appendix D
ψ_e	= Factor used to modify development length based on reinforcement coating, Sec 8.2
$\psi_{ec,N}$	= Factor used to modify tensile strength of anchors based on eccentricity of applied loads, see

	D.5.2.4, Appendix D
$\psi_{ec,V}$	= Factor used to modify shear strength of anchors based on eccentricity of applied loads, see D.6.2.5, Appendix D
$\psi_{ed,N}$	= Factor used to modify tensile strength of anchors based on proximity to edges of concrete member, see D.5.2.5, Appendix D
$\psi_{ed,V}$	= Factor used to modify shear strength of anchors based on proximity to edges of concrete member, see D.6.2.6, Appendix D
$\psi_{h,V}$	= Factor used to modify shear strength of anchors located in concrete members with $h_a < 1.5c_{a1}$, see D.6.2.8, Appendix D
ψ_s	= Factor used to modify development length based on reinforcement size, Sec 8.2
ψ_t	= Factor used to modify development length based on reinforcement location, Sec 8.2
ψ_w	= Factor used to modify development length for welded deformed wire reinforcement in tension, Sec 8.2
A	= Effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that of the reinforcement, divided by the number of bars. When the flexural reinforcement consists of different bar sizes the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar used
A_{sk}	= Area of skin reinforcement per unit height in a side face
C_t	= Factor relating shear and torsional stress properties = $\frac{b_w d}{\sum x^2 y}$
d_c	= Thickness of concrete cover measured from extreme tension fibre to centre of bar or wire located closest thereto
M_1	= Moment of resistance of a section without compression steel
M_2	= Additional moment of resistance due to added compression steel A'_s and additional tension steel a_{s2}
s	= Spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement
T_c	= Torsional moment strength provided by concrete
T_s	= Torsional moment strength provided by torsion reinforcement
x_1	= Shorter centre to centre dimension of closed rectangular stirrup
y_1	= Longer centre to centre dimension of closed rectangular stirrup
z	= Quantity limiting distribution of flexural reinforcement, see Eq (6.2.35)
α_t	= Coefficient equal to $(2 + y_1/x_1)/3$ but not more than 1.5
β_1	= Factor defined in 6.2.3.7
ε	= Time-dependent factor for sustained load
ρ_{min}	= Minimum ratio of tension reinforcement

6.1.2 General

- 6.1.2.1 Members shall be designed for adequate strength in accordance with the provisions of this chapter, using load factors specified in 2.6.5.1 and strength reduction factors ϕ in 6.2.3.1.
- 6.1.2.2 Design of reinforced concrete members using Working Stress Design method (Appendix B) is also permitted.

- 6.1.2.3 Structures and structural members shall be designed to have design strength at all sections at least equal to the required strength (U) calculated for the factored loads and forces in such combinations as are stipulated in Chapter 2, Loads. The nominal strength provided for the section multiplied by the strength reduction factor ϕ shall be equal to or greater than the calculated required strength U .
- 6.1.2.4 Members shall also meet all the other requirements of this Code to ensure adequate performance at service loads.
- 6.1.2.5 Design strength of reinforcement represented by the values of f_y and f_{yt} used in design calculations shall not exceed 550 MPa, except for prestressing steel and for transverse reinforcement in 6.3.9.3 and Sec. 8.3. f_y or f_{yt} may exceed 420 MPa, only if the ratio of the actual tensile strength to the actual yield strength is not less than 1.20, and the elongation percentage is not less than 16.
- 6.1.2.6 For structural concrete, f'_c shall not be less than 17 MPa. No maximum value of f'_c shall apply unless restricted by a specific Code provision.

6.1.3 Loading

- 6.1.3.1 Loads and their combinations shall be in accordance with the requirements specified in Chapter 2, Loads.
- 6.1.3.2 Structures shall be designed to resist all applicable loads.
- 6.1.3.3 Effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and unequal settlement of supports shall be duly considered.

6.1.4 Methods of analysis

- 6.1.4.1 Members of frames or continuous construction (beams or one-way slabs) shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified for redistribution of moments in continuous flexural members according to 6.1.5. Design is permitted to be simplified by using the assumptions specified in 6.1.6 & 6.1.9 through 6.1.12.
- 6.1.4.2 Frame analysis by approximate methods shall be permitted for buildings of usual types of construction, spans, and story heights.
- 6.1.4.3 Provided (a) through (e) below are satisfied, the approximate moments and shears given here shall be permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), as an alternate to frame analysis:
 - a) There are two or more spans;
 - b) Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent;
 - c) Loads are uniformly distributed;
 - d) Unfactored live load, L , does not exceed three times unfactored dead load, D ; and
 - e) Members are prismatic.

For calculating negative moments, l_n is taken as the average of the adjacent clear span lengths.

Positive moment

End spans

Discontinuous end unrestrained	$w_u l_n^2 / 11$
Discontinuous end integral with support	$w_u l_n^2 / 14$
Interior spans	$w_u l_n^2 / 16$
Negative moments 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 3.048 m; and beams where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of the span	$w_u l_n^2 / 12$
Negative moment at interior face of exterior support for members built integrally with supports	
Where support is 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	$w_u l_n / 2$

6.1.4.4 Strut-and-tie models, provided in Appendix A, shall be permitted to be used in the design of structural concrete.

6.1.5 Redistribution of moments in continuous flexural members

6.1.5.1 It shall be permitted to decrease factored moments calculated by elastic theory at sections of maximum negative or maximum positive moment in any span of continuous flexural members for any assumed loading arrangement by not more than $1000\varepsilon_t$ percent, with a maximum of 20 percent, except where approximate values for moments are used.

6.1.5.2 Redistribution of moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

- 6.1.5.3 At all other sections within the spans, the reduced moment shall be used for calculating redistributed moments. Static equilibrium shall have to be maintained after redistribution of moments for each loading arrangement.

6.1.6 **Span length**

- 6.1.6.1 The span length of a simply supported beam shall be taken as the smaller of the distance between the centres of bearings, or the clear distance between supports plus the effective depth.
- 6.1.6.2 For determination of moments in analysis of frames or continuous construction, span length shall be taken as the distance center-to-center of supports.
- 6.1.6.3 Design on the basis of moments at faces of support shall be permitted for beams built integrally with supports.
- 6.1.6.4 It shall be permitted to analyze solid or ribbed slabs built integrally with supports, with clear spans not more than 3 m, as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected.

6.1.7 **Modulus of elasticity**

- 6.1.7.1 Modulus of elasticity, E_c , for concrete shall be permitted to be taken as $w_c^{1.5} 0.043 \sqrt{f'_c}$ (in MPa) for values of w_c between 1440 and 2560 kg/m³. For normalweight concrete, E_c shall be permitted to be taken as $4700 \sqrt{f'_c}$.
- 6.1.7.2 Modulus of elasticity, E_s , for reinforcement shall be permitted to be taken as 200,000 MPa.

6.1.8 Lightweight concrete

- 6.1.8.1 To account for the use of lightweight concrete, unless specifically noted otherwise, a modification factor λ appears as a multiplier of $\sqrt{f'_c}$ in all applicable equations and sections of this Code, where $\lambda = 0.85$ for sand-lightweight concrete and 0.75 for all-lightweight concrete. Linear interpolation between 0.75 and 0.85 shall be permitted, on the basis of volumetric fractions, when a portion of the lightweight fine aggregate is replaced with normalweight fine aggregate. Linear interpolation between 0.85 and 1.0 shall be permitted, on the basis of volumetric fractions, for concrete containing normalweight fine aggregate and a blend of lightweight and normalweight coarse aggregates. For normalweight concrete, $\lambda = 1.0$. If average splitting tensile strength of lightweight concrete, f_{ct} , is specified, $\lambda = f_{ct}/(0.56\sqrt{f'_c}) \leq 1.0$.

6.1.9 Stiffness

- 6.1.9.1 For computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems, use of any set of reasonable assumptions shall be permitted. The assumptions adopted shall be consistent throughout analysis.
- 6.1.9.2 Both in determining moments and in design of members, effect of haunches shall be considered.

6.1.10 Effective stiffness for determining lateral deflections

- 6.1.10.1 Lateral deflections resulting from service lateral loads for reinforced concrete building systems shall be computed by either a linear analysis with member stiffness determined using 1.4 times the flexural stiffness defined in 6.1.10.2 and 6.1.10.3 or by a more detailed analysis. Member properties shall not be taken greater than the gross section properties.
- 6.1.10.2 Lateral deflections resulting from factored lateral loads for reinforced concrete building systems shall be computed either by linear analysis with member stiffness defined by (a) or (b), or by a more detailed analysis considering the reduced stiffness of all members under the loading conditions:
- a) By section properties defined in 6.3.10.4.1(a) through (c); or
 - b) 50 percent of stiffness values based on gross section properties.
- 6.1.10.3 Lateral deflections resulting from factored lateral loads shall be permitted to be computed by using linear analysis, where two-way slabs without beams are designated as part of the seismic-force-resisting system. The stiffness of slab members shall be defined by a model that is in substantial agreement with results of comprehensive tests and analysis and the stiffness of other frame members shall be as defined in 6.1.10.2.

6.1.11 Considerations for Columns

- 6.1.11.1 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 resulting the maximum ratio of moment to axial load shall also be considered.
- 6.1.11.2 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.

- 6.1.11.3 It shall be permitted to assume far ends of columns built integrally with the structure to be fixed, while computing gravity load moments in columns.
- 6.1.11.4 Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

6.1.12 Live load arrangement

- 6.1.12.1 The following shall be permitted to assume:
 - a) The live load is applied only to the floor or roof under consideration; and
 - b) The far ends of columns built integrally with the structure are considered to be fixed.
- 6.1.12.2 Arrangement of live load shall be permitted to be assumed to 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.

6.1.13 Construction of T-beam

- 6.1.13.1 In the construction of T-beam, the flange and web shall be built integrally or otherwise effectively bonded together.
- 6.1.13.2 Width of slab effective as a T-beam flange shall not exceed one-quarter of 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; and
 - b) One-half the clear distance to the next web.
- 6.1.13.3 The effective overhanging flange width for beams with a slab on one side only shall not exceed:
 - a) One-twelfth the span length of the beam;
 - b) Six times the slab thickness; and
 - c) One-half the clear distance to the next web.
- 6.1.13.4 Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one-half the width of web and an effective flange width not more than four times the width of web.
- 6.1.13.5 When primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement shall be provided in the top of the slab in the direction perpendicular to the beam and in accordance with the following:
 - 6.1.13.5.1 Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width assumed to act as a cantilever. 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.
 - 6.1.13.5.2 Spacing of transverse reinforcement shall be not farther apart than five times the slab thickness, nor farther apart than 450 mm.

6.1.14 Construction of joist

- 6.1.14.1 Construction of joist consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

- 6.1.14.2 Width of ribs shall not be less than 100 mm, and the ribs shall have a depth of not more than 3-1/2 times the minimum width of rib.
- 6.1.14.3 Clear spacing between ribs shall not exceed 750 mm.
- 6.1.14.4 Joist construction not meeting the limitations of 6.1.14.1 through 6.1.14.3 shall be designed as slabs and beams.
- 6.1.14.5 When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to f'_c in the joists are used:
 - 6.1.14.5.1 For shear and negative moment strength computations, the vertical shells of fillers in contact with the ribs shall be permitted to include. Other portions of fillers shall not be included in strength computations.
 - 6.1.14.5.2 Slab thickness over permanent fillers shall be not less than one-twelfth the clear distance between ribs, nor less than 40 mm.
 - 6.1.14.5.3 Reinforcement normal to the ribs shall be provided in the slab in one-way joists, as required by 8.1.11
- 6.1.14.6 When removable forms or fillers are used, which do not comply with 6.1.14.5, then:
 - 6.1.14.6.1 Slab thickness shall be not less than one-twelfth the clear distance between ribs, nor less than 50 mm.
 - 6.1.14.6.2 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 8.1.11
- 6.1.14.7 Where conduits or pipes as permitted by relevant provisions of embedments in concrete 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.
- 6.1.14.8 For joist construction, V_c shall be permitted to be 10 percent more than that specified in Sec 6.4.
- 6.1.15 **Separate floor finish**
 - 6.1.15.1 Unless placed monolithically with the floor slab or designed in accordance with requirements of Sec. 6.12, floor finish shall not be included as part of a structural member.
 - 6.1.15.2 All concrete floor finishes shall be permitted to be considered as part of required cover or total thickness for nonstructural considerations.

6.2 STRENGTH AND SERVICEABILITY REQUIREMENTS

6.2.1 General

- 6.2.1.1 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.

- 6.2.1.2 Members also shall meet all other requirements of this Code to ensure adequate performance at service load levels.

6.2.2 Required strength

- 6.2.2.1 Required strength U shall be at least equal to the effects of factored loads in such combinations as are stipulated in Chapter 2, Loads.
- 6.2.2.2 If resistance to impact effects is taken into account in design, such effects shall be included with L .
- 6.2.2.3 Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service.

6.2.3 Design Strength

- 6.2.3.1 Design strength provided by a member, and its connections to other members, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with the requirements and assumptions of this chapter, multiplied by a strength reduction factors ϕ as stipulated in 6.2.3.2, 6.2.3.3, and 6.2.3.4.

- 6.2.3.2 Strength reduction factor ϕ shall be as given in 6.2.3.2.1 through 6.2.3.2.6:

- 6.2.3.2.1 Tension-controlled sections as defined in 6.3.3.4 0.90

- 6.2.3.2.2 Compression-controlled sections, as defined in 6.3.3.3:

Members with spiral reinforcement conforming to 6.3.9.3..... 0.75

Other reinforced members 0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength, ϵ_t , is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as ϵ_t increases from the compression controlled strain limit to 0.005 (Also see Fig. 6.2.3.1). While interpolating, it shall be permitted to round ϕ to second digit after decimal.

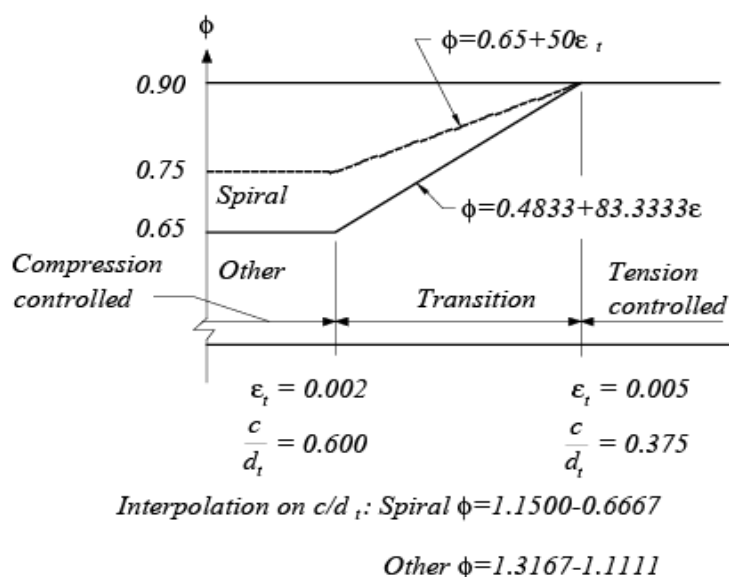


Fig. 6.2.3.1-Variation of ϕ with net tensile strain in extreme tension steel, ϵ_t and c/d_t for Grade 420 reinforcement and for prestressing steel (see sec.6.2.3.2.2)

6.2.3.2.3 It shall be permitted for compression-controlled sections, as defined in 6.3.3.3, the following optional, more conservative alternative values of strength reduction factor ϕ , where less controlled construction environment justifies such selection according to engineering judgment of the designer:

Members with spiral reinforcement conforming to 6.3.9.3	0.70
Other reinforced members	0.60

For sections in which the net tensile strain in the extreme tension steel at nominal strength, ϵ_t , is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as ϵ_t increases from the compression controlled strain limit to 0.005 (Also see Fig. 6.2.3.2). While interpolating, it shall be permitted to round ϕ to second digit after decimal.

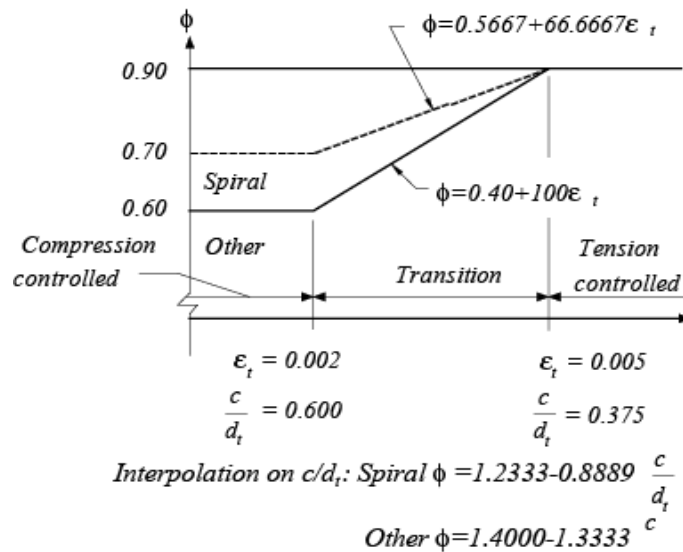


Fig. 6.2.3.2-Variation of ϕ with net tensile strain in extreme tension steel, ϵ_t and c/d_t for Grade 420 reinforcement and for prestressing steel with reduced values of ϕ (0.6 and 0.7) for compression controlled sections (see sec.6.2.3.2.3, Optional application in case of less controlled environment as per engineering judgment)

- 6.2.3.2.4 Shear and torsion 0.75
- 6.2.3.2.5 Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models: 0.65
- 6.2.3.2.6 Strut-and-tie models (Appendix A), and struts, ties, nodal zones, and bearing areas in such models: 0.75
- 6.2.3.2.7 Calculation of development length specified in Sec 8.2 does not require a strength reduction factor ϕ .
- 6.2.3.3 For structures relying on intermediate precast structural walls in Seismic Design Category D, special moment frames, or special structural walls to resist earthquake effects, E , ϕ shall be modified as given in (a) through (c):
- For any structural member that is designed to resist E , if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal flexural strength of the member, ϕ for shear shall be 0.60. The nominal flexural strength shall be determined considering the most critical factored axial loads and including E ;

- b) For diaphragms, ϕ for shear shall not exceed the minimum ϕ for shear used for the vertical components of the primary seismic-force-resisting system;
 - c) For joints and diagonally reinforced coupling beams, ϕ for shear shall be 0.85.
- 6.2.3.4 Strength reduction factor ϕ shall be 0.60 for flexure, compression, shear, and bearing of structural plain concrete.

6.2.4 Design strength for reinforcement

The values of f_y and f_{yt} used in design calculations shall not exceed 550 MPa, except for transverse reinforcement in 6.3.9.3 and Sec. 8.3.

6.2.5 Control of deflections

- 6.2.5.1 Reinforced concrete members subjected 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.
- 6.2.5.2 One-way construction (nonprestressed)
- 6.2.5.2.1 Minimum thickness stipulated in Table 6.2.5.1 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 can be used without adverse effects.
- 6.2.5.2.2 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 6.2.5.1 — MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

	Minimum thickness, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one- way slabs	$l/20$	$l/24$	$l/28$	$l/10$
Beams or ribbed one- way slabs	$l/16$	$l/18.5$	$l/21$	$l/8$

Notes:

Values given shall be used directly for members with normalweight concrete and Grade 420 reinforcement. For other conditions, the values shall be modified as follows:

- a) For lightweight concrete having equilibrium density, w_c , in the range of 1440 to 1840 kg/m³, the values shall be multiplied by $(1.65 - 0.0003w_c)$ but not less than 1.09.
- b) For f_y other than 420 MPa, the values shall be multiplied by $(0.4 + f_y/700)$.

- 6.2.5.2.3 If not stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be computed with the modulus of elasticity for concrete, E_c , as specified in 6.1.7.1 (normal weight or lightweight concrete) and with the effective moment of inertia, I_e , as follows, but not greater than I_g

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \quad (6.2.1)$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (6.2.2)$$

and

$$f_r = 0.62 \lambda \sqrt{f'_c} \quad (6.2.3)$$

6.2.5.2.4 I_e shall be permitted to be taken for continuous members as the average of values obtained from Eq. (6.2.1) for the critical positive and negative moment sections. For prismatic members, I_e shall be permitted to be taken as the value obtained from Eq. (6.2.1) at midspan for simple and continuous spans, and at support for cantilevers.

6.2.5.2.5 If the values are not obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members (normalweight or lightweight concrete) shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor λ_Δ

$$\lambda_\Delta = \frac{\xi}{1 + 50 \rho'} \quad (6.2.4)$$

where ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. It shall be permitted to assume ξ , the time-dependent factor for sustained loads, to be equal to:

5 years or more	2.0
12 months	1.4
6 months	1.2
3 months	1.0

6.2.5.2.6 The value of deflection computed in accordance with 6.2.5.2.2 through 6.2.5.2.5 shall not exceed limits stipulated in Table 6.2.5.2.

6.2.5.3 Two-way construction (nonprestressed)

6.2.5.3.1 The minimum thickness of slabs or other two-way construction designed in accordance with the provisions of Sec. 6.5 and conforming with the requirements of 6.5.6.1.2 shall be governed by Section 6.2.5.3. The thickness of slabs without interior beams spanning between the supports on all sides shall satisfy the requirements of 6.2.5.3.2 or 6.2.5.3.4. The thickness of slabs with beams spanning between the supports on all sides shall satisfy requirements of 6.2.5.3.3 or 6.2.5.3.4.

6.2.5.3.2 If slabs are without interior beams spanning between the supports and have a ratio of long to short span not greater than 2, the minimum thickness shall be in accordance with the provisions of Table 6.2.5.3 and shall not be less than the following values:

Slabs without drop panels as defined in 6.5.2.5	125 mm;
Slabs with drop panels as defined in 6.5.2.5	100 mm.

TABLE 6.2.5.2 — MAXIMUM ALLOWABLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$l / 180$ *

Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$l/360$
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-term deflection due to all sustained loads and the immediate deflection due to any additional live load) [†]	$l/480$ [‡]
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$l/240$ [§]

*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-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

[†]Long-term deflection shall be determined in accordance with 6.2.5.2.5, 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.

[‡]Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

[§]Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded

TABLE 6.2.5.3—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

f_y , MPa [†]	Without drop panels [‡]			With drop panels [‡]		
	Exterior panels		Interior	Exterior panels		Interior
	Without edge beams	With edge beams [§]		Without edge beams	With edge beams [§]	
280	$l_n/33$	$l_n/36$	$l_n/36$	$l_n/36$	$l_n/40$	$l_n/40$
420	$l_n/30$	$l_n/33$	$l_n/33$	$l_n/33$	$l_n/36$	$l_n/36$
520	$l_n/28$	$l_n/31$	$l_n/31$	$l_n/31$	$l_n/34$	$l_n/34$

*For two-way construction, l_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.

[†]For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.

[‡]Drop panels as defined in 6.5.2.5.

[§]Slabs with beams between columns along exterior edges. The value of α_r for the edge beam shall not be less than 0.8.

6.2.5.3.3 The minimum thickness, h for slabs with beams spanning between the supports on all sides, shall be as follows:

- For α_{fm} equal to or less than 0.2, the provisions of 6.2.5.3.2 shall apply;
- For α_{fm} greater than 0.2 but not greater than 2.0, h shall not be less than

$$h = \frac{\ell_n \left(0.8 + \frac{f_y}{1400} \right)}{36 + 5\beta(\alpha_{fm} - 0.2)} \quad (6.2.5)$$

and not less than 125 mm;

- c) For α_{fm} greater than 2.0, h shall not be less than

$$h = \frac{\ell_n \left(0.8 + \frac{f_y}{1400} \right)}{36 + 9\beta} \quad (6.2.6)$$

and not less than 90 mm;

- d) An edge beam with a stiffness ratio α_f not less than 0.80 shall be provided at discontinuous edges, or the minimum thickness required by Eq. (6.2.5) or (6.2.6) shall be increased by at least 10 percent in the panel with a discontinuous edge.

Term ℓ_n in (b) and (c) is length of clear span in long direction measured face-to-face of beams.

Term β in (b) and (c) is ratio of clear spans in long to short direction of slab.

- 6.2.5.3.4 When computed deflections do not exceed the limits of Table 6.2.5.2, slab thickness less than the minimum required by 6.2.5.3.1, 6.2.5.3.2, and 6.2.5.3.3 shall be permitted. Deflections shall be computed taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges. The modulus of elasticity of concrete, E_c , shall be as specified in 6.1.7.1. The effective moment of inertia, I_e , shall be that given by Eq. (6.2.1); other values shall be permitted to be used if they result in computed deflections in reasonable agreement with results of comprehensive tests. Additional long-term deflection shall be computed in accordance with 6.2.5.2.5.

6.2.5.4 Composite construction

6.2.5.4.1 *Shored construction*

Where composite flexural members are supported during construction so that, after removal of temporary supports, dead load is resisted by the full composite section, it shall be permitted to consider the composite member equivalent to a monolithically cast member for computation of deflection. For nonprestressed members, the portion of the member in compression shall determine whether values in Table 6.2.5.1 for normalweight or lightweight concrete shall apply. If deflection is computed, account shall be taken of curvatures resulting from differential shrinkage of precast and cast-in-place components, and of axial creep effects in a prestressed concrete member.

6.2.5.4.2 *Unshored construction*

When the thickness of a nonprestressed precast flexural member meets the requirements of Table 6.2.5.1, deflection need not be computed. If the thickness of a nonprestressed composite member meets the requirements of Table 6.2.5.1, it is not required to compute deflection occurring after the member becomes composite, but the long-term deflection of the precast member shall be investigated for magnitude and duration of load prior to beginning of effective composite action.

- 6.2.5.4.3 The computed deflection in accordance with 6.2.5.4.1 or 6.2.5.4.2 shall not exceed limits stipulated in Table 6.2.5.2.

6.3 AXIAL LOADS AND FLEXURE

6.3.1 Scope

The provisions of Sec. 6.3 shall be applicable to the design of members subject to flexure or axial loads or a combination thereof.

6.3.2 Design assumptions

- 6.3.2.1 The assumptions given in 6.3.2.2 through 6.3.2.7, and satisfaction of applicable conditions of equilibrium and compatibility of strains shall form the basis of strength design of members for flexure and axial loads.
- 6.3.2.2 It shall be assumed that strain in reinforcement and concrete is directly proportional to the distance from the neutral axis, except that, for deep beams as defined in 6.3.7.1, an analysis that considers a nonlinear distribution of strain shall be used. Alternatively, it shall be permitted to use a strut-and-tie model. See 6.3.7, 6.4.6, and Appendix A.
- 6.3.2.3 It shall be assumed that the maximum usable strain at extreme concrete compression fiber is equal to 0.003.
- 6.3.2.4 For stress in reinforcement below f_y , it shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y , stress in reinforcement shall be considered independent of strain and equal to f_y .
- 6.3.2.5 In axial and flexural calculations of reinforced concrete, the tensile strength of concrete shall be neglected.
- 6.3.2.6 The relationship between concrete compressive stress distribution and concrete strain shall 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.
- 6.3.2.7 An equivalent rectangular concrete stress distribution defined by 6.3.2.7.1 through 6.3.2.7.3 below shall satisfy the requirements of 6.3.2.6.
- 6.3.2.7.1 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 fiber of maximum compressive strain.
- 6.3.2.7.2 Distance from the fiber of maximum strain to the neutral axis, c , shall be measured in a direction perpendicular to the neutral axis.
- 6.3.2.7.3 For f'_c between 17 and 28 MPa, β_1 shall be taken as 0.85. For f'_c above 28 MPa, β_1 shall be reduced linearly at a rate of 0.05 for each 7 MPa of strength in excess of 28 MPa, but β_1 shall not be taken less than 0.65. For f'_c between 28 and 56 MPa, β_1 may be calculated from Eq. (6.3.1).

$$\beta_1 = 0.85 - 0.007143(f'_c - 28) \text{ and } 0.65 \leq \beta_1 \leq 0.85 \quad (6.3.1)$$

6.3.3 General principles and requirements

- 6.3.3.1 Stress and strain compatibility using assumptions in 6.3.2 shall be the basis for design of cross sections subject to flexure or axial loads, or a combination thereof.
- 6.3.3.2 A cross section shall be considered to be in balanced strain conditions when the tension reinforcement reaches the strain corresponding to f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.
- 6.3.3.3 Sections are compression-controlled if the net tensile strain in the extreme tension steel, ϵ_t , is equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed strain limit of 0.003 (Fig.6.3.3.1). The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 420 reinforcement, it shall be permitted to set the compression-controlled strain limit equal to 0.002. For other grades compression-controlled strain limit may be determined by dividing the yield strength by modulus of elasticity E and then rounding the value obtained to four significant digits after the decimal. For example, for Grade 500 reinforcement, the compression-controlled strain limit shall equal to 0.0025.

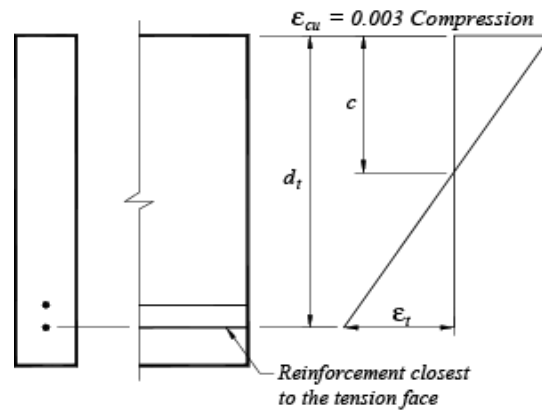


Fig. 6.3.3.1—Strain distribution and net tensile strain

- 6.3.3.4 Sections are tension-controlled if the net tensile strain in the extreme tension steel, ε_t , is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with ε_t between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.
- 6.3.3.5 Net tensile strain in the extreme tension steel at nominal strength, ε_t shall not be less than 0.004 for nonprestressed flexural members and nonprestressed members with factored axial compressive load less than $0.10f'_cA_g$
- 6.3.3.5.1 Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase the strength of flexural members.
- 6.3.3.6 For compression members, design axial strength ϕP_n shall not be taken greater than $\phi P_{n,max}$, computed by Eq. (6.3.2) or (6.3.3).
- 6.3.3.6.1 For nonprestressed members with spiral reinforcement conforming to Sec. 8.1 or composite members conforming to 6.3.13:
- $$\phi P_{n,max} = 0.85\phi[0.85f'_c(A_g - A_{st}) + f_y A_{st}] \quad (6.3.2)$$
- 6.3.3.6.2 For nonprestressed members with tie reinforcement conforming to Sec. 8.1:
- $$\phi P_{n,max} = 0.80\phi[0.85f'_c(A_g - A_{st}) + f_y A_{st}] \quad (6.3.3)$$
- 6.3.3.7 Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial force P_u at given eccentricity shall not exceed that given in 6.3.3.6. The maximum factored moment M_u shall be magnified for slenderness effects in accordance with 6.3.10.

6.3.4 Spacing of lateral supports for flexural members

- 6.3.4.1 Distance between lateral supports for a beam shall not exceed 50 times b , the least width of compression flange or face.
- 6.3.4.2 Effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

6.3.5 Minimum reinforcement for members in flexure

- 6.3.5.1 At every section of a flexural member where tensile reinforcement is required by analysis, except as provided in 6.3.5.2, 6.3.5.3, and 6.3.5.4, A_s provided shall not be less than that given by

$$A_{s,min} = \frac{0.25\sqrt{f'_c}}{f_y} b_w d \quad (6.3.4)$$

and not less than $1.4b_w d/f_y$.

- 6.3.5.2 For statically determinate members with a flange in tension, $A_{s,min}$ shall not be less than the value given by Eq. (6.3.4), except that b_w is replaced by either $2b_w$ or the width of the flange, whichever is smaller.
- 6.3.5.3 If, at every section, A_s provided is at least one-third greater than that required by analysis, the requirements of 6.3.5.1 and 6.3.5.2 need not be applied.
- 6.3.5.4 For structural slabs and footings of uniform thickness, $A_{s,min}$ in the direction of the span shall be the same as that required by 8.1.11. Maximum spacing of this reinforcement shall not exceed three times the thickness, nor 450 mm.

6.3.6 Distribution of flexural reinforcement in one-way slabs and beams

- 6.3.6.1 Rules for distribution of flexural reinforcement to control flexural cracking in beams and in one-way slabs (slabs reinforced to resist flexural stresses in only one direction) are prescribed in this section.
- 6.3.6.2 Distribution of flexural reinforcement in two-way slabs shall be as required by 6.5.3.
- 6.3.6.3 As prescribed in 6.3.6.4, flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section.
- 6.3.6.4 The spacing of reinforcement closest to the tension face, s , shall be less than that given by

$$s = 380 \left(\frac{280}{f_s} \right) - 2.5c_c \quad (6.3.5)$$

but shall not exceed $300(280/f_s)$, where c_c is the least distance from surface of reinforcement to the tension face. If there is only one bar or wire nearest to the extreme tension face, s used in Eq. (6.3.5) is the width of the extreme tension face.

Calculated stress f_s in reinforcement closest to the tension face at service load shall be computed based on the unfactored moment. It shall be permitted to take f_s as $2/3f_y$.

- 6.3.6.5 For structures subject to very aggressive exposure or designed to be watertight, provisions of 6.3.6.4 are not sufficient. For such structures, special investigations and precautions are required.
- 6.3.6.6 When 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 6.1.13, or a width equal to one-tenth the span, whichever is smaller. If the effective flange width exceeds one-tenth the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.
- 6.3.6.7 Longitudinal skin reinforcement shall be uniformly distributed along both side faces of a member (Fig. 6.3.6.1), where h of a beam or joist exceeds 900 mm. Skin reinforcement shall extend for a distance $h/2$ from the tension face. The spacing s shall be as provided in 6.3.6.4, where c_c is the least distance from the surface of the skin reinforcement to the side face. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires.

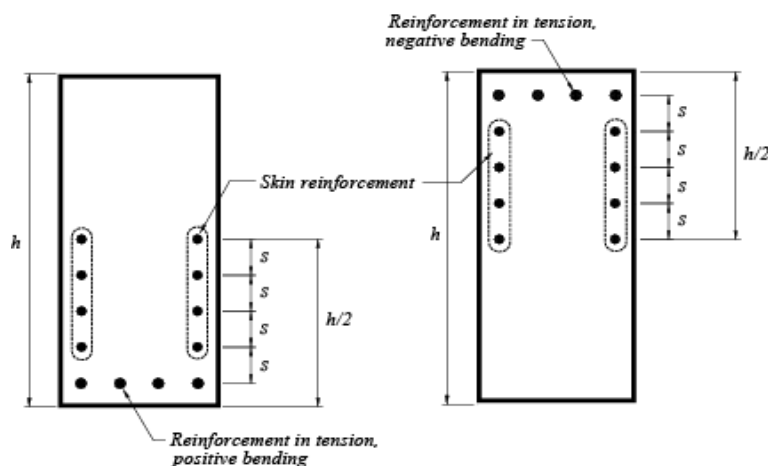


Fig. 6.3.6.1—Skin reinforcement for beams and joists with $h > 900$ mm.

6.3.7 Deep beams

6.3.7.1 Deep beams are members loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports, and have either:

- clear spans, l_n , equal to or less than four times the overall member depth; or
- regions with concentrated loads within twice the member depth from the face of the support.

Deep beams shall be designed either taking into account nonlinear distribution of strain, or by Appendix A. (See also 6.4.6.1 and 8.2.7.6) Lateral buckling shall be considered.

6.3.7.2 V_n of deep beams shall be in accordance with 6.4.6.

6.3.7.3 Minimum area of flexural tension reinforcement, $A_{s,min}$, shall conform to 6.3.5.

6.3.7.4 Minimum horizontal and vertical reinforcement in the side faces of deep beams shall satisfy either A.3.3 or 6.4.6.4 and 6.4.6.5.

6.3.8 Design dimensions for compression members

6.3.8.1 Isolated compression member with multiple spirals

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 8.1.7.

6.3.8.2 Monolithically built compression member 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.

6.3.8.3 Equivalent circular compression member replacing other shapes

In lieu of using the full gross area for design of a compression member with a square, octagonal, or other shaped cross section, it shall be permitted to use 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.

6.3.8.4 Limits of section

For a compression member with a cross section larger than required by considerations of loading, it shall be permitted to base the minimum reinforcement and strength on a reduced effective area A_g not less

than one-half the total area. This provision shall not apply to special moment frames or special structural walls designed in accordance with Sec. 8.3.

6.3.9 Limits of reinforcement for compression members

6.3.9.1 For noncomposite compression members, the area of longitudinal reinforcement, A_{st} , shall be not less than $0.01A_g$ or more than $0.06A_g$. To avoid practical difficulties in placing and compacting of concrete as well as to deliver ductility to noncomposite compression members, area of longitudinal reinforcement, A_{st} , is preferred not to exceed $0.04A_g$ unless absolutely essential.

6.3.9.2 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 6.3.9.3.

6.3.9.3 Volumetric spiral reinforcement ratio, ρ_s , shall be not less than the value given by

$$\rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \quad (6.3.6)$$

where the value of f_{yt} used in Eq. (6.3.6) shall not exceed 700 MPa. For f_{yt} greater than 420 MPa, lap splices according to 8.1.9.3(e) shall not be used.

6.3.10 Slenderness effects in compression members

6.3.10.1 Slenderness effects shall be permitted to be neglected in the following cases:

a) for compression members not braced against sidesway when:

$$\frac{k\ell_u}{r} \leq 22 \quad (6.3.7)$$

b) for compression members braced against sidesway when:

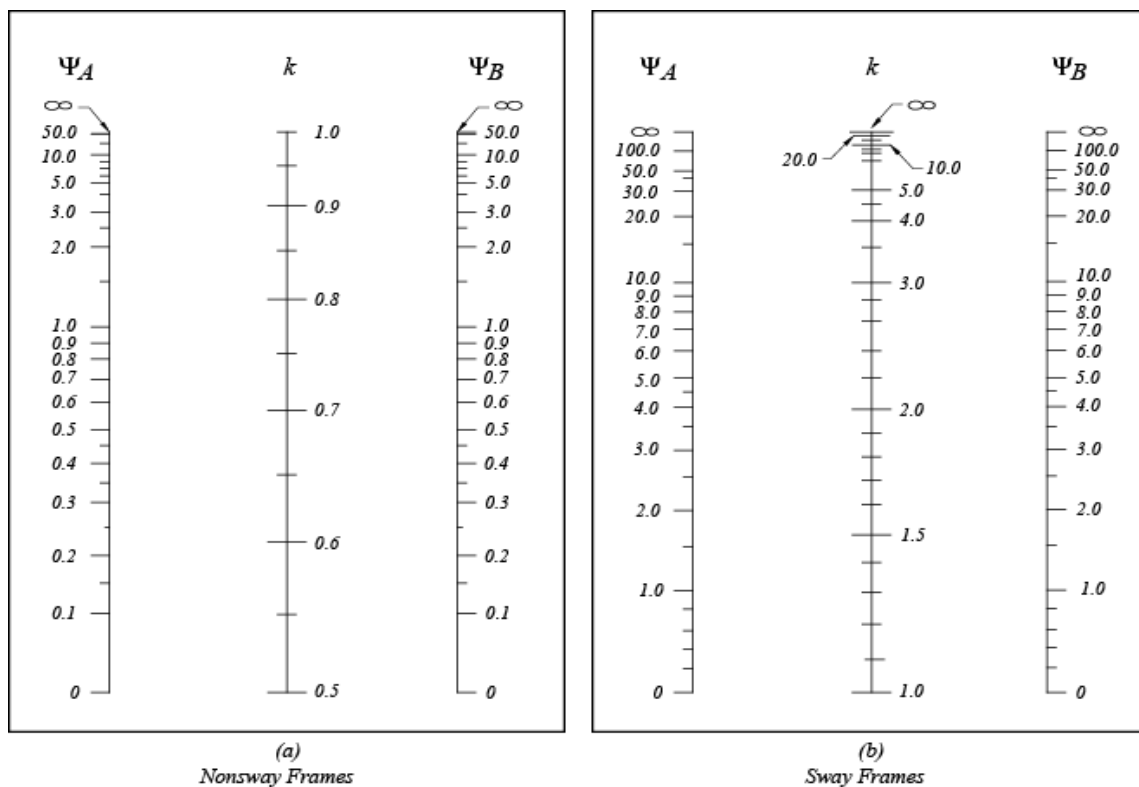
$$\frac{k\ell_u}{r} \leq 34 - 12(M_1/M_2) \leq 40 \quad (6.3.8)$$

where M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature.

Compression members may be considered to be braced against sidesway when bracing elements have a total stiffness, resisting lateral movement of that story, of at least 12 times the gross stiffness of the columns within the story.

The Jackson and Moreland Alignment Charts (Fig. 6.3.10.1), which allow a graphical determination of k for a column of constant cross section in a multibay frame may be used as the primary design aid to estimate the effective length factor k .

- 6.3.10.1.1 The unsupported length of a compression member, l_u , shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support in the direction being considered. Where column capitals or haunches are present, l_u shall be measured to the lower extremity of the capital or haunch in the plane considered.
- 6.3.10.1.2 It shall be permitted to take the radius of gyration, r , 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, it shall be permitted to compute r for the gross concrete section.



Ψ = ratio of $\Sigma(EI/\ell_c)$ of compression members to $\Sigma(EI/\ell)$ of flexural members in plane at one end of a compression member

ℓ = span length of flexural member measured center to center of joints

Fig. 6.3.10.1—Effective length factors k .

6.3.10.2 When slenderness effects are not neglected as permitted by 6.3.10.1, the design of compression members, restraining beams, and other supporting members shall be based on the factored forces and moments from a second-order analysis satisfying 6.3.10.3, 6.3.10.4, or 6.3.10.5. These members shall also satisfy 6.3.10.2.1 and 6.3.10.2.2. The dimensions of each member cross section used in the analysis shall be within 10 percent of the dimensions of the members shown on the design drawings or the analysis shall be repeated.

6.3.10.2.1 Total moment including second-order effects in compression members, restraining beams, or other structural members shall not exceed 1.4 times the moment due to first-order effects.

6.3.10.2.2 Second-order effects shall be considered along the length of compression members. It shall be permitted to account for these effects using the moment magnification procedure outlined in 6.3.10.6.

6.3.10.3 Nonlinear second-order analysis

Second-order analysis shall consider material nonlinearity, member curvature and lateral drift, duration of loads, shrinkage and creep, and interaction with the supporting foundation. The analysis procedure shall have been shown to result in prediction of strength in substantial agreement with results of comprehensive tests of columns in statically indeterminate reinforced concrete structures.

6.3.10.4 Elastic second-order analysis

Elastic second-order analysis shall consider section properties determined taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and the effects of load duration.

6.3.10.4.1 It shall be permitted to use the following properties for the members in the structure:

- a) Modulus of elasticity E_c from 6.1.7.1
- b) Moments of inertia, I

Compression members:

Columns	$0.70I_g$
Walls —Uncracked	$0.70I_g$
—Cracked	$0.35I_g$

Flexural members:

Beams	$0.35I_g$
Flat plates and flat slabs	$0.25I_g$
c) Area	$1.0A_g$

Alternatively, the moments of inertia of compression and flexural members, I , shall be permitted to be computed as follows:

Compression members:

$$I = \left(0.80 + 25 \frac{A_{st}}{A_g} \right) \left(1 - \frac{M_u}{P_u h} - 0.5 \frac{P_u}{P_o} \right) I_g \leq 0.875 I_g \quad (6.3.9)$$

where P_u and M_u shall be determined from the particular load combination under consideration, or the combination of P_u and M_u determined in the smallest value of I . I need not be taken less than $0.35I_g$.

Flexural members:

$$I = (0.10 + 25\rho) \left(1.2 - 0.2 \frac{b_w}{d} \right) I_g \leq 0.5 I_g \quad (6.3.10)$$

For continuous flexural members, I shall be permitted to be taken as the average of values obtained from Eq. (6.3.10) for the critical positive and negative moment sections. I need not be taken less than $0.25I_g$. The cross-sectional dimensions and reinforcement ratio used in the above formulas shall be within 10 percent of the dimensions and reinforcement ratio shown on the design drawings or the stiffness evaluation shall be repeated.

- 6.3.10.4.2 When sustained lateral loads are present, I for compression members shall be divided by $(1 + \beta_{ds})$. The term β_{ds} shall be taken as the ratio of maximum factored sustained shear within a story to the maximum factored shear in that story associated with the same load combination, but shall not be taken greater than 1.0.

6.3.10.5 Procedure for moment magnification

Columns and stories in structures shall be designated as nonsway or sway columns or stories. The design of columns in nonsway frames or stories shall be based on 6.3.10.6. The design of columns in sway frames or stories shall be based on 6.3.10.7.

- 6.3.10.5.1 A column in a structure shall be permitted to be assumed as nonsway if the increase in column end moments due to second-order effects does not exceed 5 percent of the first-order end moments.

- 6.3.10.5.2 A story within a structure is permitted to be assumed as nonsway if:

$$Q = \frac{\sum P_u \Delta_o}{V_{us} \ell_c} \leq 0.05 \quad (6.3.11)$$

where $\sum P_u$ and V_{us} are the total factored vertical load and the horizontal story shear, respectively, in the story being evaluated, and Δ_o is the first-order relative lateral deflection between the top and the bottom of that story due to V_{us} .

6.3.10.6 Procedure for moment magnification — Nonsway

Compression members shall be designed for factored axial force P_u and the factored moment amplified for the effects of member curvature M_c where

$$M_c = \delta_{ns} M_2 \quad (6.3.12)$$

where

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \geq 1.0 \quad (6.3.13)$$

and

$$P_c = \frac{\pi^2 EI}{(k \ell_u)^2} \quad (6.3.14)$$

- 6.3.10.6.1 EI shall be taken as

$$EI = \frac{(0.2E_c I_g + E_s I_{se})}{1 + \beta_{dns}} \quad (6.3.15)$$

or

$$EI = \frac{0.4E_c I_g}{1 + \beta_{dns}} \quad (6.3.16)$$

As an alternative, EI shall be permitted to be computed using the value of I from Eq. (6.3.9) divided by $(1 + \beta_{dns})$.

6.3.10.6.2 The term β_{dns} shall be taken as the ratio of maximum factored axial sustained load to maximum factored axial load associated with the same load combination, but shall not be taken greater than 1.0.

6.3.10.6.3 The effective length factor, k , shall be permitted to be taken as 1.0.

6.3.10.6.4 For members with no transverse load between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} \quad (6.3.17)$$

where M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

6.3.10.6.5 Factored moment, M_2 , in Eq. (6.3.12) shall not be taken less than

$$M_{2,min} = P_u(15 + 0.03h) \quad (6.3.18)$$

about each axis separately, where 0.6 and h are in mm. For members in which $M_{2,min}$ exceeds M_2 , the value of C_m in Eq. (6.3.17) shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments, M_1/M_2 .

6.3.10.7 Procedure for moment magnification — Sway

Moments M_1 and M_2 at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \quad (6.3.19)$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \quad (6.3.20)$$

where δ_s is computed according to 6.3.10.7.3 or 6.3.10.7.4.

6.3.10.7.1 Flexural members shall be designed for the total magnified end moments of the compression members at the joint.

6.3.10.7.2 The values of E_c and I given in 6.3.10.4 shall be used for determining the effective length factor k and it shall not be less than 1.0.

6.3.10.7.3 The moment magnifier δ_s shall be calculated as

$$\delta_s = \frac{1}{1-Q} \geq 1 \quad (6.3.21)$$

If δ_s calculated by Eq. (6.3.21) exceeds 1.5, δ_s shall be calculated using second-order elastic analysis or 6.3.10.7.4.

6.3.10.7.4 Alternatively, it shall be permitted to calculate δ_s as

$$\delta_s = \frac{1}{1 - \frac{\sum P_u}{0.75 \sum P_c}} \geq 1 \quad (6.3.22)$$

where $\sum P_u$ is the summation for all the factored vertical loads in a story and $\sum P_c$ is the summation for all sway-resisting columns in a story. P_c is calculated using Eq. (6.3.14) with k determined from 6.3.10.7.2 and EI from 6.3.10.6.1.

6.3.11 Axially loaded members supporting slab system

Axially loaded members supporting a slab system included within the scope of 6.5.1 shall be designed as provided in Sec. 6.3 and in accordance with the additional requirements of Sec. 6.5.

6.3.12 Column load transmission through floor system

If f'_c of a column is greater than 1.4 times that of the floor system, transmission of load through the floor system shall be provided by 6.3.12.1, 6.3.12.2, or 6.3.12.3.

- 6.3.12.1 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 2 ft into the slab from face of column. Column concrete shall be well integrated with floor concrete, and shall be placed in accordance with relevant provisions for construction joints of columns, walls etc. with beams, slabs etc. To avoid accidental placing of lower strength concrete in the columns, the structural designer shall indicate on the drawing where the high and low strength concretes are to be placed.
- 6.3.12.2 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.
- 6.3.12.3 For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength. In the application of 6.3.12.3, the ratio of column concrete strength to slab concrete strength shall not be taken greater than 2.5 for design.

6.3.13 Composite compression members

- 6.3.13.1 All members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars shall be included in composite compression members.
- 6.3.13.2 A composite member strength shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.
- 6.3.13.3 Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.
- 6.3.13.4 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.
- 6.3.13.5 For evaluation of slenderness effects, radius of gyration, r , of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_{sx}}{(E_c A_g / 5) + E_s A_{sx}}} \quad (6.3.23)$$

and, as an alternative to a more accurate calculation, EI in Eq. (6.3.14) shall be taken either as Eq. (6.3.15) or

$$EI = \frac{(E_c I_g / 5)}{1 + \beta_d} + E_s I_{sx} \quad (6.3.24)$$

6.3.13.6 Concrete core encased by structural steel

- 6.3.13.6.1 When a composite member is a structural steel encased concrete core, the thickness of the steel encasement shall be not less than $b \sqrt{\frac{f_y}{3E_s}}$ for each face of width b nor $b \sqrt{\frac{f_y}{8E_s}}$ for circular sections of diameter h
- 6.3.13.6.2 When computing A_{sx} and I_{sx} , longitudinal bars located within the encased concrete core shall be permitted to be used.

6.3.13.7 Spiral reinforcement around structural steel core

A composite member with spirally reinforced concrete around a structural steel core shall conform to 6.3.13.7.1 through 6.3.13.7.4.

6.3.13.7.1 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 350 MPa.

6.3.13.7.2 Spiral reinforcement shall conform to 6.3.9.3.

6.3.13.7.3 Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.06 times net area of concrete section.

6.3.13.7.4 Longitudinal bars located within the spiral shall be permitted to be used in computing A_{sx} and I_{sx} .

6.3.13.8 Tie reinforcement around structural steel core

Laterally tied concrete around a structural steel core forming a composite member shall conform to 6.3.13.8.1 through 6.3.13.8.7.

6.3.13.8.1 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 350 MPa.

6.3.13.8.2 Lateral ties shall extend completely around the structural steel core.

6.3.13.8.3 Lateral ties shall have a diameter not less than 0.02 times the greatest side dimension of composite member, except that ties shall not be smaller than $\phi 10$ mm and are not required to be larger than $\phi 16$ mm. Welded wire reinforcement of equivalent area shall be permitted.

6.3.13.8.4 Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 0.5 times the least side dimension of the composite member.

6.3.13.8.5 Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.06 times net area of concrete section.

6.3.13.8.6 A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than onehalf the least side dimension of the composite member.

6.3.13.8.7 Longitudinal bars located within the ties shall be permitted to be used in computing A_{sx} and I_{sx} .

6.3.14 Bearing strength

6.3.14.1 Design bearing strength of concrete shall not exceed $\phi(0.85f'_cA_1)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but by not more than 2 (Fig. 6.3.14.1).

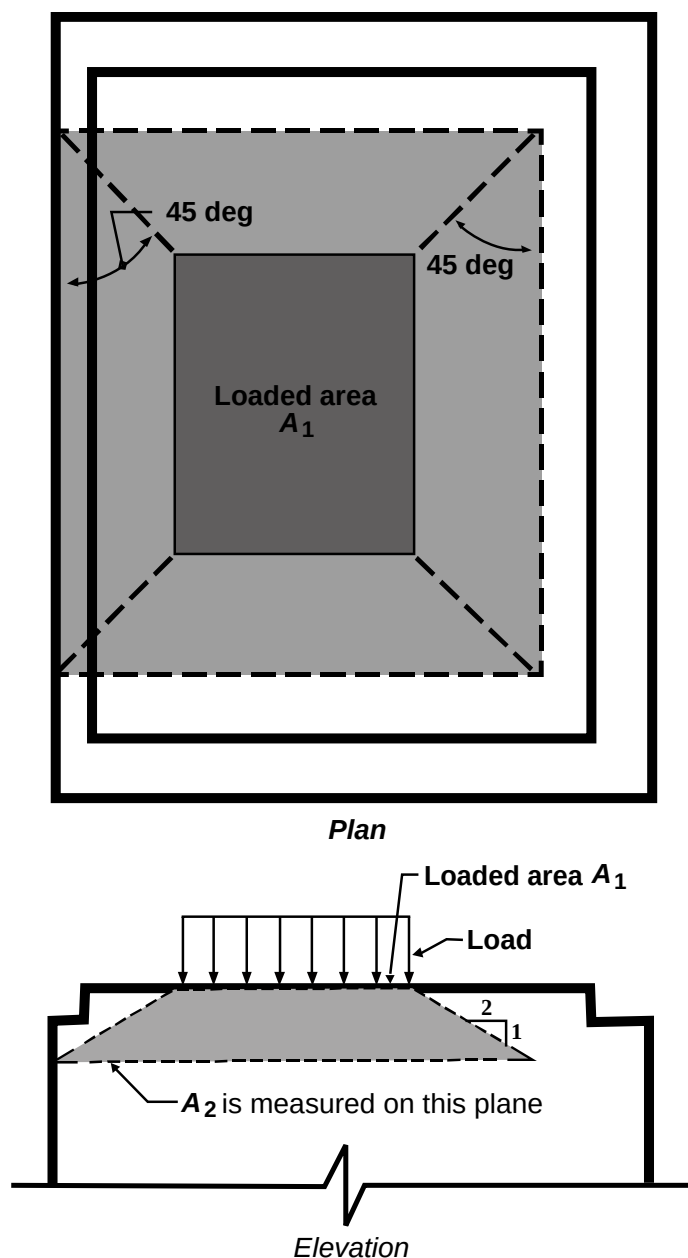


Fig. 6.3.14.1 - Determination of area A_2 in stepped or sloped supports using frustum (6.3.14.1).

6.3.15 Design for Flexure

6.3.15.1 Design of Rectangular Beams

- a) Formula for singly reinforced beams : The following equations which are based on the simplified stress block of 6.3.2.7, are applicable to singly reinforced rectangular beams along with T-beams where the neutral axis lies within the flange.

$$A_s = \frac{M_n}{f_y(d-a/2)} \quad (6.3.25)$$

where

$$a = \frac{A_s f_y}{0.85 f'_c b} \quad (6.3.26)$$

By estimating an initial value of a , Eq (6.3.25) can be used to determine an approximate value of A_s . That value can be substituted in Eq (6.3.26) to get a better estimate of a and hence a new $(d - a/2)$ can be determined for substitution in Eq (6.3.25).

In Eq (6.3.25), nominal flexural strength of section, M_n may be taken as factored moment at section, M_u divided by strength reduction factor, $\phi = 0.9$ as a preliminary value. A_s determined from Eq (6.3.25) shall have to give a reinforcement ratio, $\rho = A_s/bd$ not exceeding ρ_{max} , where

$$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.004} \quad (6.3.27)$$

Above, $\epsilon_u = 0.003$

Additionally, A_s determined from Eq (6.3.25) shall have to satisfy the requirements of minimum reinforcement for members in flexure as per 6.3.5.

Revised ϕ shall be determined from 6.2.3.2 based on either $c/d_t = a/\beta_1 d_t$ or ϵ_t , where ϵ_t is the net tensile strain in the reinforcement furthest from the compression face of the concrete at the depth d_t . Strain, ϵ_t may be calculated from Eq. (6.3.27) by replacing 0.004 by ϵ_t and ρ_{max} by ρ .

- b) Design formulae for doubly reinforced beams: A doubly reinforced beam shall be designed only when there is a restriction on depth of beam and maximum tensile reinforcement allowed cannot produce the required moment M_u .

To establish if doubly reinforced beam is required the following approach can be followed:

Determine,

$$\rho_{0.005} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} \quad (6.3.28)$$

$$A_s = \rho_{0.005} bd$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right) \quad (6.3.29)$$

If ϕM_n is less than required moment M_u with $\phi = 0.9$, a doubly reinforced beam is needed and then taking values of A_s and ϕM_n from above, put

$$A_{s1} = A_s \text{ and}$$

$$\phi M_{n1} = \phi M_n$$

Then, the following values are to be evaluated,

$$\phi M_{n2} = M_u - \phi M_{n1} \quad (6.3.30)$$

$$A_{s2} = \frac{\phi M_{n2}}{\phi f_y (d - d')}$$

Assuming compression steel yields (needs to be checked later),

$$A'_s = A_{s2}$$

$$A_s = A_{s1} + A_{s2}$$

Check $\rho \geq \bar{\rho}_{cy}$ for compression steel yielding, where

$$\bar{\rho}_{cy} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{d'}{d} \frac{\epsilon_u}{\epsilon_u - \epsilon_y} + \rho' \quad (6.3.31)$$

If $\rho \geq \bar{\rho}_{cy}$ (i.e. compression steel yields),

find $a = \frac{(A_s - A'_s)f_y}{0.85f'_c b}$ and find c , ε_t and confirm $\varphi = 0.9$ in the above equations. Value of φ shall be determined from 6.2.3.2 based on either $c/d_t = a/\beta_1 d_t$ or ε_t , as stated above for rectangular beams.

If compression steel does not yield, c is to be found from concrete section force equilibrium condition, $C=T$ which will result in a quadratic equation of c . f'_s needs to be calculated from strain diagram and A'_s revised.

$$A'_s = A_{s2} \frac{f_y}{f'_s}$$

$$A_s = A_{s1} + A_{s2}$$

ε_t shall be calculated from c for finding φ .

6.3.15.2 Design of T-Beams

a) General:

For effective widths and other parameters for T-, L- or isolated beams, 6.1.13.2 to 6.1.13.4 shall apply.

b) Formulae for T-beams : A T-beam shall be treated as a rectangular beam if $a \leq h_f$ where a is obtained from Eq (6.3.26). In using Eq (6.3.26), if A_s is not known, it may be initially assumed as :

$$A_s = \frac{M_n}{f_y(d - h_f/2)} \quad (6.3.32)$$

If a , thus obtained, is greater than h_f the beam shall be considered as a T-beam, in which case the following formulae shall be applicable :

$$A_{sf} = \frac{0.85f'_c(b - b_w)h_f}{f_y} \quad (6.3.33)$$

$$M_{n1} = A_{sf}f_y(d - h_f/2) \quad (6.3.34)$$

$$M_{n2} = M_n - M_{n1} \quad (6.3.35)$$

$$A_s - A_{sf} = \frac{M_{n2}}{f_y(d - a/2)} \quad (6.3.36)$$

$$\text{and } a = \frac{(A_s - A_{sf})f_y}{0.85f'_c b_w} \quad (6.3.37)$$

By estimating an initial value of a , Eq (6.3.36) can be used to obtain an approximate value of $(A_s - A_{sf})$. That value of $(A_s - A_{sf})$ can be substituted in Eq (6.3.37) to get a better estimate of a .

Net tensile strain requirements will be satisfied as long as depth to neutral axis, $c \leq 0.429 d_t$. This will occur if

$$\rho_w < \rho_{w,max}$$

$$\text{Where, } \rho_w = \frac{A_s}{b_w d} \quad (6.3.38)$$

$$\text{And, } \rho_{w,max} = \rho_{max} + \rho_f \quad (6.3.39)$$

$$\text{Again, } \rho_f = \frac{A_{sf}}{b_w d} \quad (6.3.40)$$

And ρ_{max} is as defined by Eq. (6.3.27). For c/d_t ratios between 0.429 and 0.375, equivalent to ρ_w between the $\rho_{w,max}$ from Eq. (6.3.39) and $\rho_{w,max}$ calculated by substituting ρ from Eq. (6.3.27) with 0.005 in place of 0.004 and ρ for ρ_{max} , the strength reduction factor, ϕ must be adjusted for ε_t in accordance with 6.2.3.2.

6.4 SHEAR AND TORSION

6.4.1 Shear strength

- 6.4.1.1 Except for members designed in accordance with Appendix A, design of cross sections subject to shear shall be based on

$$\phi V_n \geq V_u \quad (6.4.1)$$

where V_u is the factored shear force at the section considered and V_n is nominal shear strength computed by

$$V_n = V_c + V_s \quad (6.4.2)$$

where V_c is nominal shear strength provided by concrete calculated in accordance with 6.4.2, or 6.4.10, and V_s is nominal shear strength provided by shear reinforcement calculated in accordance with 6.4.3, 6.4.8.9, or 6.4.10.

- 6.4.1.1.1 The effect of any openings in members shall be considered in determining V_n .
- 6.4.1.1.2 In evaluating 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 shall be permitted to be included.
- 6.4.1.2 Except as allowed in 6.4.1.2.1, the values of $\sqrt{f'_c}$ used in this chapter shall not exceed 8.3 MPa.
- 6.4.1.2.1 Values of $\sqrt{f'_c}$ greater than 8.3 MPa shall be permitted in computing V_c , V_{ci} , and V_{cw} for reinforced concrete beams and concrete joist construction having minimum web reinforcement in accordance with 6.4.3.5.3, or 6.4.4.5.2.
- 6.4.1.3 Computation of maximum V_u at supports in accordance with 6.4.1.3.1 shall be permitted if all conditions (a), (b), and (c) are satisfied:
- Support reaction, in direction of applied shear, introduces compression into the end regions of member;
 - Loads are applied at or near the top of the member;
 - No concentrated load occurs between face of support and location of critical section defined in 6.4.1.3.1.

6.4.1.3.1 Sections located less than a distance d from face of support shall be permitted to be designed for V_u computed at a distance d .

6.4.1.4 For deep beams, brackets and corbels, walls, and slabs and footings, the special provisions of 6.4.6 through 6.4.10 shall apply.

6.4.2 Contribution of concrete to shear strength

6.4.2.1 V_c shall be computed by provisions of 6.4.2.1.1 through 6.4.2.1.3, unless a more detailed calculation is made in accordance with 6.4.2.2. Throughout this chapter, except in 6.4.5, λ shall be as defined in 6.1.8.1.

6.4.2.1.1 For members subject to shear and flexure only,

$$V_c = 0.17\lambda \sqrt{f'_c} b_w d \quad (6.4.3)$$

6.4.2.1.2 For members subject to axial compression,

$$V_c = 0.17(1 + \frac{N_u}{14A_g})\lambda \sqrt{f'_c} b_w d \quad (6.4.4)$$

Quantity N_u/A_g shall be expressed in MPa.

6.4.2.1.3 For members subject to significant axial tension, V_c shall be taken as zero unless a more detailed analysis is made using 6.4.2.2.3.

6.4.2.2 V_c shall be permitted to be computed by the more detailed calculation of 6.4.2.2.1 through 6.4.2.2.3.

6.4.2.2.1 For members subject to shear and flexure only,

$$V_c = (0.16\lambda \sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}) b_w d \quad (6.4.5)$$

but not greater than $0.29\lambda \sqrt{f'_c} b_w d$. When computing V_c by Eq. (6.4.5), $V_u d/M_u$ shall not be taken greater than 1.0, where M_u occurs simultaneously with V_u at section considered.

6.4.2.2.2 For members subject to axial compression, it shall be permitted to compute V_c using Eq. (6.4.5) with M_m substituted for M_u and $V_u d/M_u$ not then limited to 1.0, where

$$M_m = M_u - N_u \frac{(4h-d)}{8} \quad (6.4.6)$$

However, V_c shall not be taken greater than

$$V_c = 0.29\lambda \sqrt{f'_c} b_w d \sqrt{1 + \frac{0.29N_u}{A_g}} \quad (6.4.7)$$

N_u/A_g shall be expressed in MPa. When M_m as computed by Eq. (6.4.6) is negative, V_c shall be computed by Eq. (6.4.7).

6.4.2.2.3 For members subject to significant axial tension,

$$V_c = 0.17(1 + \frac{0.29N_u}{A_g})\lambda \sqrt{f'_c} b_w d \quad (6.4.8)$$

but not less than zero, where N_u is negative for tension. N_u/A_g shall be expressed in MPa.

6.4.2.3 For circular members, the area used to compute V_c shall be taken as the product of the diameter and effective depth of the concrete section. It shall be permitted to take d as 0.80 times the diameter of the concrete section.

6.4.3 Shear strength contribution of reinforcement

6.4.3.1 Types of shear reinforcement

6.4.3.1.1

The following types of shear reinforcement shall be permitted:

- a) Stirrups perpendicular to axis of member;
- b) Welded wire reinforcement with wires located perpendicular to axis of member;
- c) Spirals, circular ties, or hoops.
- d) Stirrups making an angle of 45 degrees or more with longitudinal tension reinforcement;
- e) Longitudinal reinforcement with bent portion making an angle of 30 degrees or more with the longitudinal tension reinforcement;
- f) Combinations of stirrups and bent longitudinal reinforcement.

6.4.3.2

The values of f_y and f_{yt} used in design of shear reinforcement shall not exceed 420 MPa, except the value shall not exceed 550 MPa for welded deformed wire reinforcement.

6.4.3.3

Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be developed at both ends according to 8.2.10.

6.4.3.4 Limits in spacing for shear reinforcement

6.4.3.4.1

Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$, nor 600 mm.

6.4.3.4.2

The spacing of inclined stirrups and bent longitudinal reinforcement shall be such that every 45-degree 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.

6.4.3.4.3

Where V_s exceeds $0.33\sqrt{f'_c}b_wd$, maximum spacings given in 6.4.3.4.1 and 6.4.3.4.2 shall be reduced by one-half.

6.4.3.5 Minimum shear reinforcement

6.4.3.5.1

A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members, where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

- a) Footings and solid slabs;
- b) Hollow-core units with total untopped depth not greater than 315 mm and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;
- c) Concrete joist construction defined by 6.1.14;
- d) Beams with h not greater than 250 mm;
- e) Beam integral with slabs with h not greater than 600 mm and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

- f) Beams constructed of steel fiber-reinforced, normalweight concrete with f'_c not exceeding 40 MPa, h not greater than 600 mm, and V_u not greater than $\phi 0.17 \sqrt{f'_c} b_w d$.

6.4.3.5.2

Minimum shear reinforcement requirements of 6.4.3.5.1 shall be permitted to be waived if shown by test that required M_n and V_n 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.

6.4.3.5.3

Where shear reinforcement is required by 6.4.3.5.1 or for strength and where 6.4.4.1 allows torsion to be neglected, $A_{v,min}$ shall be computed by

$$A_{v,min} = 0.062 \sqrt{f'_c} \frac{b_w s}{f_{yt}} \quad (6.4.9)$$

but shall not be less than $(0.35 b_w s) / f_{yt}$.

6.4.3.6 Design of shear reinforcement

- 6.4.3.6.1 Where V_u exceeds ϕV_c , shear reinforcement shall be provided to satisfy Eq. (6.4.1) and (6.4.2), where V_s shall be computed in accordance with 6.4.3.6.2 through 6.4.3.6.9.

- 6.4.3.6.2 Where shear reinforcement perpendicular to axis of member is used,

$$V_s = \frac{A_v f_{yt} d}{s} \quad (6.4.10)$$

where A_v is the area of shear reinforcement within spacing s .

- 6.4.3.6.3 Where circular ties, hoops, or spirals are used as shear reinforcement, V_s shall be computed using Eq. (6.4.10) where d is defined in 6.4.2.3 for circular members, A_v shall be taken as two times the area of the bar in a circular tie, hoop, or spiral at a spacing s , s is measured in a direction parallel to longitudinal reinforcement, and f_{yt} is the specified yield strength of circular tie, hoop, or spiral reinforcement.

- 6.4.3.6.4 Where inclined stirrups are used as shear reinforcement,

$$V_s = \frac{A_v f_{yt} (\sin \alpha + \cos \alpha) d}{s} \quad (6.4.11)$$

where α is angle between inclined stirrups and longitudinal axis of the member, and s is measured in direction parallel to longitudinal reinforcement.

- 6.4.3.6.5 Where shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support,

$$V_s = A_v f_y \sin \alpha \quad (6.4.12)$$

but not greater than $0.25 \sqrt{f'_c} b_w d$, where α is angle between bent-up reinforcement and longitudinal axis of the member.

- 6.4.3.6.6 Where shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, V_s shall be computed by Eq. (6.4.11).
- 6.4.3.6.7 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.
- 6.4.3.6.8 Where more than one type of shear reinforcement is used to reinforce the same portion of a member, V_s shall be computed as the sum of the values computed for the various types of shear reinforcement.
- 6.4.3.6.9 V_s shall not be taken greater than $0.66 \sqrt{f'_c} b_w d$.

6.4.4 Design for torsion

Design for torsion shall be in accordance with 6.4.4.1 through 6.4.4.6. A beam subjected to torsion is idealized as a thin-walled tube with the core concrete cross section in a solid beam neglected as shown in Fig. 6.4.4.1.

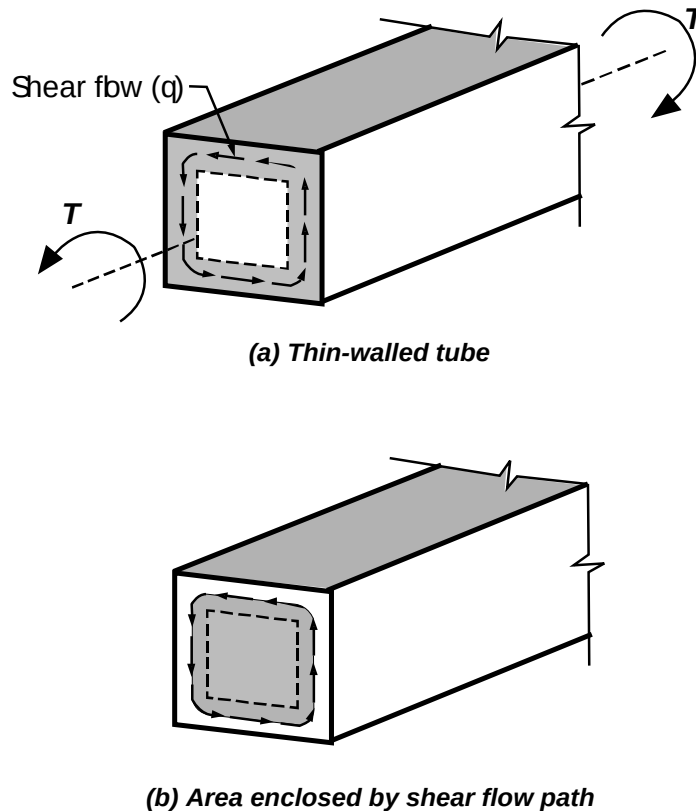


Fig. 6.4.4.1- (a) Torsional resistance by thin-walled tube (6.4.4); (b) Ineffective inner area enclosed by shear flow path (6.4.4)

6.4.4.1 Threshold torsion

It shall be permitted to neglect torsion effects if the factored torsional moment T_u is less than:

- a) For members not subjected to axial tension or compression

$$\phi 0.083 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right)$$

- b) For members subjected to an axial compressive or tensile force

$$\phi 0.083 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33 A_g \lambda \sqrt{f'_c}}}$$

The overhanging flange width used in computing A_{cp} and p_{cp} for members cast monolithically with a slab shall conform to 6.5.2.4. For a hollow section, A_g shall be used in place of A_{cp} in 6.4.4.1, and the outer boundaries of the section shall conform to 6.5.2.4.

6.4.4.1.1 For members cast monolithically with a slab and for isolated members with flanges, the overhanging flange width used to compute A_{cp} and p_{cp} shall conform to 6.5.2.4, except that the overhanging flanges shall be neglected in cases where the parameter A_{cp}^2/p_{cp} calculated for a beam with flanges is less than that computed for the same beam ignoring the flanges.

6.4.4.2 Evaluation of factored torsional moment

6.4.4.2.1 If the factored torsional moment, T_u , in a member is required to maintain equilibrium (Fig. 6.4.4.2) and exceeds the minimum value given in 6.4.4.1, the member shall be designed to carry T_u in accordance with 6.4.4.3 through 6.4.4.6.

6.4.4.2.2 In a statically indeterminate structure where reduction of the torsional moment in a member can occur due to redistribution of internal forces upon cracking (Fig. 6.4.4.3), the maximum T_u shall be permitted to be reduced to the values given in (a), or (b) as applicable:

a) For members, at the sections described in 6.4.4.2.4 and not subjected to axial tension or compression

$$\phi 0.33 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right)$$

b) For members subjected to an axial compressive or tensile force

$$\phi 0.33 \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{p_{cp}} \right) \sqrt{1 + \frac{N_u}{0.33 A_g \lambda \sqrt{f'_c}}}$$

In (a), or (b), the correspondingly redistributed bending moments and shears in the adjoining members shall be used in the design of these members. For hollow sections, A_{cp} shall not be replaced with A_g in 6.4.4.2.2.

- 6.4.4.2.3 It shall be permitted to take the torsional loading from a slab as uniformly distributed along the member, if not determined by a more exact analysis.
- 6.4.4.2.4 Sections located closer than a distance d from the face of a support shall be designed for not less than T_u computed at a distance d . If a concentrated torque occurs within this distance, the critical section for design shall be at the face of the support.

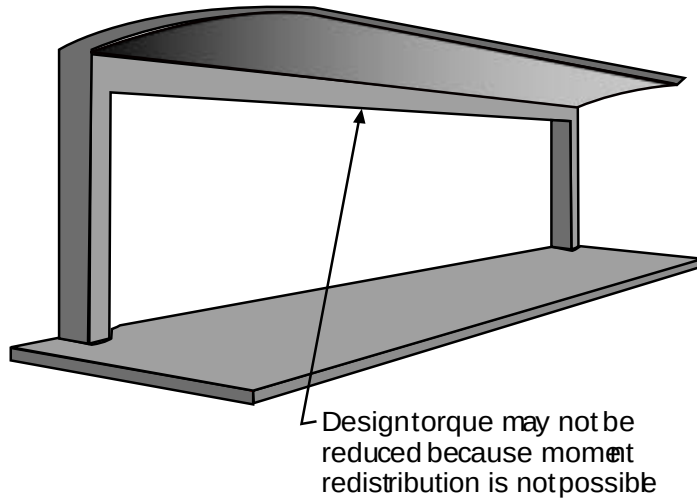


Fig. 6.4.4.2 – Design torque may not be reduced (6.4.4.2.1)

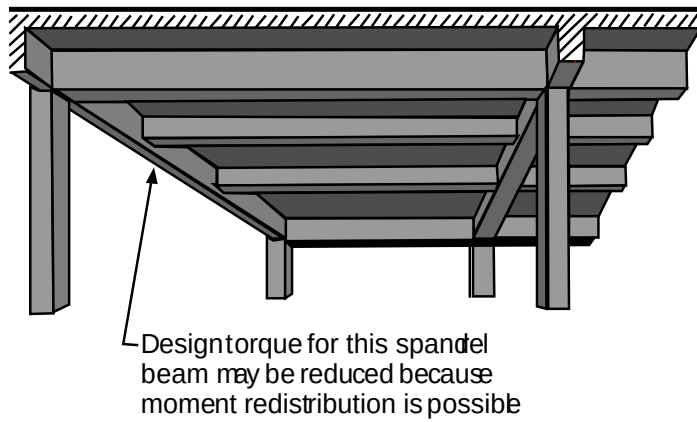


Fig. 6.4.4.3 – Design torque may be reduced (6.4.4.2.2)

- 6.4.4.3 Torsional moment strength
- 6.4.4.3.1 The cross-sectional dimensions shall be such that:

a) For solid sections

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \leq \phi \left(\frac{V_c}{b_w d} + 0.66 \sqrt{f'_c} \right) \quad (6.4.13)$$

b) For hollow sections

$$\left(\frac{V_u}{b_w d} \right) + \left(\frac{T_u P_h}{1.7 A_{oh}^2} \right) \leq \phi \left(\frac{V_c}{b_w d} + 0.66 \sqrt{f'_c} \right) \quad (6.4.14)$$

Superposition of shear stresses due to shear and torsion in hollow sections given by the left side of the inequality 6.4.14 is illustrated by Fig. 6.4.4.4(a) and that in solid sections given by the left side of the inequality 6.4.13 is illustrated by Fig. 6.4.4.4(b).

6.4.4.3.2 If the wall thickness varies around the perimeter of a hollow section, Eq. (6.4.14) shall be evaluated at the location where the left-hand side of Eq. (6.4.14) is a maximum.

6.4.4.3.3 If the wall thickness is less than A_{oh}/p_h , the second term in Eq. (6.4.14) shall be taken as

$$\left(\frac{T_u}{1.7A_{oh}t} \right)$$

where t is the thickness of the wall of the hollow section at the location where the stresses are being checked.

6.4.4.3.4 The values of f_y and f_{yt} used for design of torsional reinforcement shall not exceed 420 MPa.

6.4.4.3.5 Where T_u exceeds the threshold torsion, design of the cross section shall be based on

$$\phi T_n \geq T_u \quad (6.4.15)$$

6.4.4.3.6 T_n shall be computed by

$$T_n = \frac{2A_o A_t f_{yt}}{s} \cot \theta \quad (6.4.16)$$

where A_o shall be determined by analysis except that it shall be permitted to take A_o equal to $0.85A_{oh}$; θ shall not be taken smaller than 30 degrees nor larger than 60 degrees. It shall be permitted to take θ equal to 45 degrees.

6.4.4.3.7 The additional area of longitudinal reinforcement to resist torsion, A_l , shall not be less than

$$A_l = \frac{A_t}{s} p_h \left(\frac{f_{yt}}{f_y} \right) \cot^2 \theta \quad (6.4.17)$$

where θ shall be the same value used in Eq. (6.4.16) and A_t/s shall be taken as the amount computed from Eq. (6.4.16) not modified in accordance with 6.4.4.5.2 or 6.4.4.5.3; f_{yt} refers to closed transverse torsional reinforcement, and f_y refers to longitudinal torsional reinforcement.

- 6.4.4.3.8 Reinforcement required for torsion shall be added to that required for the shear, moment, and axial force that act in combination with the torsion. The most restrictive requirements for reinforcement spacing and placement shall be met.
- 6.4.4.3.9 It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to $M_u / (0.9df_y)$, where M_u occurs at the section simultaneously with T_u , except that the reinforcement provided shall not be less than that required by 6.4.4.5.3 or 6.4.4.6.2.

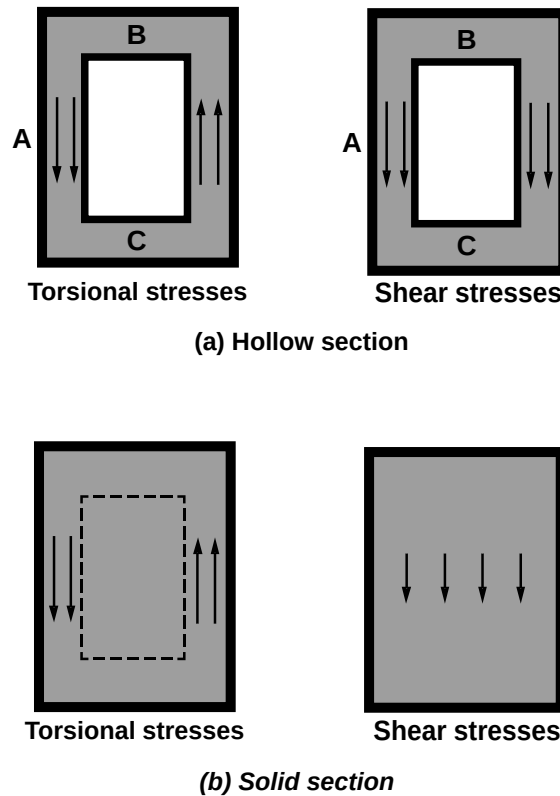


Fig. 6.4.4.4 – Superposition of torsional and shear stresses (6.4.4.3.1)

- 6.4.4.4 Details of torsional reinforcement
- 6.4.4.4.1 Torsion reinforcement shall consist of longitudinal bars or tendons and one or more of the following:
- Closed stirrups or closed ties, perpendicular to the axis of the member;
 - A closed cage of welded wire reinforcement with transverse wires perpendicular to the axis of the member;
 - Spiral reinforcement.
- 6.4.4.4.2 Transverse torsional reinforcement shall be anchored by one of the following:
- A 135-degree standard hook, or seismic hook as defined in 8.1.1(d), around a longitudinal bar;
 - According to 8.2.10.2a, 8.2.10.2b, or 8.2.10.2c in regions where the concrete surrounding the anchorage is restrained against spalling by a flange or slab or similar member.

6.4.4.4.3 Longitudinal torsion reinforcement shall be developed at both ends.

6.4.4.4.4 For hollow sections in torsion, the distance from the centerline of the transverse torsional reinforcement to the inside face of the wall of the hollow section shall not be less than $0.5A_{oh}/p_h$.

6.4.4.5 Minimum torsion reinforcement

6.4.4.5.1 A minimum area of torsional reinforcement shall be provided in all regions where T_u exceeds the threshold torsion given in 6.4.4.1.

6.4.4.5.2 Where torsional reinforcement is required by 6.4.4.5.1, the minimum area of transverse closed stirrups shall be computed by

$$A_v + 2A_t = 0.062 \sqrt{f'_c} \frac{b_w s}{f_{yt}} \quad (6.4.18)$$

but shall not be less than $(0.35b_w s)/f_{yt}$.

6.4.4.5.3 Where torsional reinforcement is required by 6.4.4.5.1, the minimum total area of longitudinal torsional reinforcement, $A_{l,min}$, shall be computed by

$$A_{l,min} = \frac{0.42\sqrt{f'_c} A_{cp}}{f_y} - \left(\frac{A_t}{s}\right) p_h \left(\frac{f_{yt}}{f_y}\right) \quad (6.4.19)$$

where A_t/s shall not be taken less than $0.175b_w/f_{yt}$; f_{yt} refers to closed transverse torsional reinforcement, and f_y refers to longitudinal reinforcement.

6.4.4.6 Spacing of torsion reinforcement

6.4.4.6.1 The spacing of transverse torsion reinforcement shall not exceed the smaller of $p_h/8$ or 300 mm.

6.4.4.6.2 The longitudinal reinforcement required for torsion shall be distributed around the perimeter of the closed stirrups with a maximum spacing of 300 mm. The longitudinal bars shall be inside the stirrups. There shall be at least one longitudinal bar in each corner of the stirrups. Longitudinal bars shall have a diameter at least 0.042 times the stirrup spacing, but not less than $\Phi 10$ mm.

6.4.4.6.3 Torsional reinforcement shall be provided for a distance of at least $(b_t + d)$ beyond the point required by analysis.

6.4.5 Shear-friction

6.4.5.1 Application of provisions of 6.4.5 shall be for cases 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.

6.4.5.2 Design of cross sections subject to shear transfer as described in 6.4.5.1 shall be based on Eq. (6.4.1), where V_n is calculated in accordance with provisions of 6.4.5.3 or 6.4.5.4.

6.4.5.3 A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement A_{vf} across the shear plane shall be designed using either 6.4.5.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

6.4.5.3.1 Provisions of 6.4.5.5 through 6.4.5.10 shall apply for all calculations of shear transfer strength.

6.4.5.4 Design method for shear-friction

6.4.5.4.1 Where shear-friction reinforcement is perpendicular to the shear plane, V_n shall be computed by

$$V_n = A_{vf} f_y \mu \quad (6.4.20)$$

where μ is coefficient of friction in accordance with 6.4.5.4.3.

6.4.5.4.2 Where shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement (Fig. 6.4.5.1), V_n shall be computed by

$$V_n = A_{vf} f_y (\mu \sin \alpha + \cos \alpha) \quad (6.4.21)$$

where α is angle between shear-friction reinforcement and shear plane.

6.4.5.4.3 The coefficient of friction μ in Eq. (6.4.20) and Eq. (6.4.21) shall be taken as:

Concrete placed monolithically	1.4 λ
Concrete placed against hardened concrete with surface intentionally roughened as specified in 6.4.5.9	1.0 λ
Concrete placed against hardened concrete not intentionally roughened	0.6 λ
Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 6.4.5.10)	0.7 λ

where $\lambda = 1.0$ for normalweight concrete and 0.75 for all lightweight concrete. Otherwise, λ shall be determined based on volumetric proportions of lightweight and normalweight aggregates as specified in 6.1.8.1, but shall not exceed 0.85.

- 6.4.5.5 For normalweight concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in 6.4.5.9, V_n shall not exceed the smallest of $0.2f'_cA_c$, $(3.3 + 0.08f'_c)A_c$ and $11A_c$, where A_c is area of concrete section resisting shear transfer. For all other cases, V_n shall not exceed the smaller of $0.2f'_cA_c$ or $5.5A_c$. Where concretes of different strengths are cast against each other, the value of f'_c used to evaluate V_n shall be that of the lower-strength concrete.
- 6.4.5.6 The value of f_y used for design of shear-friction reinforcement shall not exceed 420 MPa.
- 6.4.5.7 Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane shall be permitted to be taken as additive to $A_{vf}f_y$, the force in the shear-friction reinforcement, when calculating required A_{vf} .
- 6.4.5.8 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop f_y on both sides by embedment, hooks, or welding to special devices.
- 6.4.5.9 For the purpose of 6.4.5, 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 6 mm.
- 6.4.5.10 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.

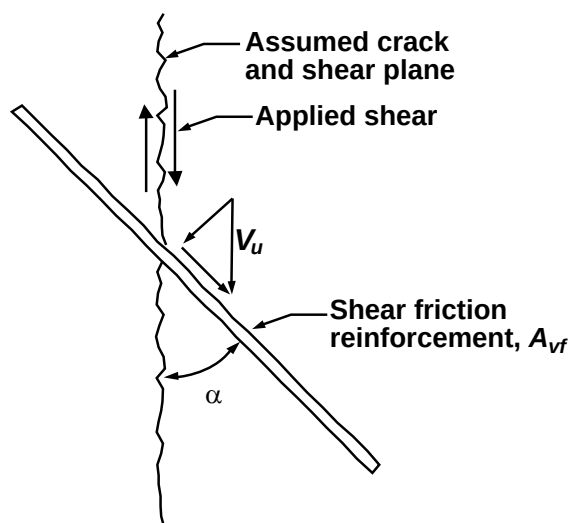


Fig. 6.4.5.1 – Shear-friction reinforcement at an angle to assumed crack (6.4.5.4.2).

6.4.6 Deep beams

- 6.4.6.1 The provisions of 6.4.6 shall apply to members with l_n not exceeding four times the overall member depth or regions of beams with concentrated loads within twice the member depth from the support that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and supports. See also 8.2.7.6.
- 6.4.6.2 Deep beams shall be designed using either nonlinear analysis as permitted in 6.3.7.1, or Appendix A.
- 6.4.6.3 V_n for deep beams shall not exceed $0.83\sqrt{f'_c}b_wd$.
- 6.4.6.4 The area of shear reinforcement perpendicular to the flexural tension reinforcement, A_v , shall not be less than $0.0025b_ws$, and s shall not exceed the smaller of $d/5$ and 300 mm.
- 6.4.6.5 The area of shear reinforcement parallel to the flexural tension reinforcement, A_{vh} , shall not be less than $0.0015b_ws_2$, and s_2 shall not exceed the smaller of $d/5$ and 300 mm.
- 6.4.6.6 It shall be permitted to provide reinforcement satisfying A.3.3 instead of the minimum horizontal and vertical reinforcement specified in 6.4.6.4 and 6.4.6.5.

6.4.7 Provisions for brackets and corbels

- 6.4.7.1 Brackets and corbels (Fig. 6.4.7.1 & Fig. 6.4.7.2) with a shear span-to-depth ratio a_v/d less than 2 shall be permitted to be designed using Appendix A. Design shall be permitted using 6.4.7.3 and 6.4.7.4 for brackets and corbels with:
- a_v/d not greater than 1, and
 - subject to factored horizontal tensile force, N_{uc} , not larger than V_u .

The requirements of 6.4.7.2, 6.4.7.3.2.1, 6.4.7.3.2.2, 6.4.7.5, 6.4.7.6, and 6.4.7.7 shall apply to design of brackets and corbels. Effective depth d shall be determined at the face of the support.

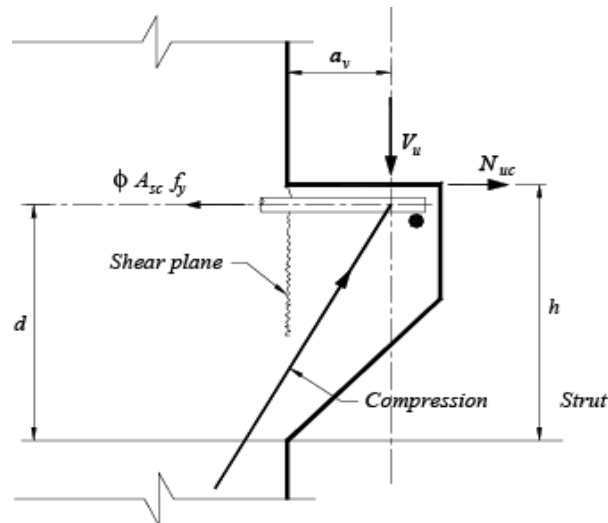


Fig. 6.4.7.1—Structural action of a corbel

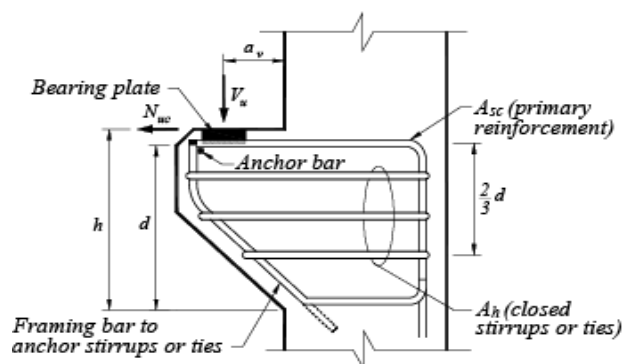


Fig. 6.4.7.2—Notation used in Section 6.4.7

- 6.4.7.2 Depth at outside edge of bearing area shall not be less than $0.5d$.
- 6.4.7.3 Section at face of support shall be designed to resist simultaneously V_u , a factored moment $[V_u a_v + N_{uc}(h - d)]$, and a factored horizontal tensile force, N_{uc} .
- 6.4.7.3.1 In all design calculations in accordance with 6.4.7, ϕ shall be taken equal to 0.75.
- 6.4.7.3.2 Design of shear-friction reinforcement, A_{vf} , to resist V_u shall be in accordance with 6.4.5.
- 6.4.7.3.2.1 For normal weight concrete, V_n shall not exceed the smallest of $0.2f'_c b_w d$, $(3.3 + 0.08f'_c)b_w d$, and $11b_w d$.
- 6.4.7.3.2.2 For all-lightweight or sand-lightweight concrete, V_n shall not be taken greater than the smaller of $(0.2 - 0.07a_v/d)f'_c b_w d$ and $(5.5 - 1.9a_v/d)b_w d$.
- 6.4.7.3.3 Reinforcement A_f to resist factored moment $[V_u a_v + N_{uc}(h - d)]$ shall be computed in accordance with 6.3.2 and 6.3.3.
- 6.4.7.3.4 Reinforcement A_n to resist factored tensile force N_{uc} shall be determined from $\phi A_n f_y \geq N_{uc}$. Factored tensile force, N_{uc} , shall not be taken less than $0.2V_u$ unless provisions are made to avoid tensile forces. N_{uc} shall be regarded as a live load even if tension results from restraint of creep, shrinkage, or temperature change.
- 6.4.7.3.5 Area of primary tension reinforcement A_{sc} shall not be less than the larger of $(A_f + A_n)$ and $(2A_{vf}/3 + A_n)$.
- 6.4.7.4 Total area, A_h , of closed stirrups or ties parallel to primary tension reinforcement shall not be less than $0.5(A_{sc} - A_n)$. Distribute A_h uniformly within $(2/3)d$ adjacent to primary tension reinforcement.
- 6.4.7.5 A_{sc}/bd shall not be less than $0.04(f'_c/f_y)$.
- 6.4.7.6 At front face of bracket or corbel, primary tension reinforcement shall be anchored by one of the following:
- By a structural weld to a transverse bar of at least equal size; weld to be designed to develop f_y of primary tension reinforcement;
 - By bending primary tension reinforcement back to form a horizontal loop; or
 - By some other means of positive anchorage.

6.4.7.7

Bearing area on bracket or corbel shall not project beyond straight portion of primary tension reinforcement, nor project beyond interior face of transverse anchor bar (if one is provided).

6.4.8 Provisions for walls

6.4.8.1 Design of walls for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 6.4.10. Design for horizontal in-plane shear forces in a wall shall be in accordance with 6.4.8.2 through 6.4.8.9. Alternatively, it shall be permitted to design walls with a height not exceeding two times the length of the wall for horizontal shear forces in accordance with Appendix A and 6.4.8.9.2 through 6.4.8.9.5.

6.4.8.2 Design of horizontal section for shear in plane of wall shall be based on Eq. (6.4.1) and (6.4.2), where V_c shall be in accordance with 6.4.8.5 or 6.4.8.6 and V_s shall be in accordance with 6.4.8.9.

6.4.8.3 V_n at any horizontal section for shear in plane of wall shall not be taken greater than $0.83 \sqrt{f'_c} h d$, where h is thickness of wall, and d is defined in 6.4.8.4.

6.4.8.4 For design for horizontal shear forces in plane of wall, d shall be taken equal to $0.8l_w$. A larger value of d , equal to the distance from extreme compression fiber to center of force of all reinforcement in tension, shall be permitted to be used when determined by a strain compatibility analysis.

6.4.8.5 If a more detailed calculation is not made in accordance with 6.4.8.6, V_c shall not be taken greater than $0.17\lambda \sqrt{f'_c} h d$ for walls subject to axial compression, or V_c shall not be taken greater than the value given in 6.4.2.2.3 for walls subject to axial tension.

6.4.8.6 V_c shall be permitted to be the lesser of the values computed from Eq. (6.4.22) and (6.4.23)

$$V_c = 0.27\lambda \sqrt{f'_c} h d + \frac{N_u d}{4l_w} \quad (6.4.22)$$

or

$$V_c = \left[0.05\lambda \sqrt{f'_c} + \frac{l_w(0.1\lambda \sqrt{f'_c} + 0.2 \frac{N_u}{l_w h})}{\frac{M_u}{V_u} \frac{l_w}{2}} \right] h d \quad (6.4.23)$$

where l_w is the overall length of the wall, and N_u is positive for compression and negative for tension. If $(M_u/V_u - l_w/2)$ is negative, Eq. (6.4.23) shall not apply.

6.4.8.7 Sections located closer to wall base than a distance $l_w/2$ or one-half the wall height, whichever is less, shall be permitted to be designed for the same V_c as that computed at a distance $l_w/2$ or one-half the height.

6.4.8.8 Where V_u is less than $0.5\phi V_c$, reinforcement shall be provided in accordance with 6.4.8.9 or in accordance with Sec. 6.6. Where V_u exceeds $0.5\phi V_c$, wall reinforcement for resisting shear shall be provided in accordance with 6.4.8.9.

6.4.8.9 Design of shear reinforcement for walls

6.4.8.9.1 Where V_u exceeds ϕV_c , horizontal shear reinforcement shall be provided to satisfy Eq. (6.4.1) and (6.4.2), where V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s} \quad (6.4.24)$$

where A_v is area of horizontal shear reinforcement within spacing s , and d is determined in accordance with 6.4.8.4. Vertical shear reinforcement shall be provided in accordance with 6.4.8.9.4.

- 6.4.8.9.2 Ratio of horizontal shear reinforcement area to gross concrete area of vertical section, ρ_t shall not be less than 0.0025.
- 6.4.8.9.3 Spacing of horizontal shear reinforcement shall not exceed the smallest of $l_w/5$, $3h$, and 450 mm, where l_w is the overall length of the wall.
- 6.4.8.9.4 Ratio of vertical shear reinforcement area to gross concrete area of horizontal section, $\rho_l\rho$ shall not be less than the larger of
- $$\rho_l = 0.0025 + 0.5 \left(2.5 - \frac{h_w}{l_w} \right) (\rho_t - 0.0025) \quad (6.4.25)$$
- and 0.0025, The value of $\rho_l\rho$ calculated by Eq. (6.4.25) need not be greater than ρ_t required by 6.4.8.9.1. In Eq. (6.4.25), l_w is the overall length of the wall, and h_w is the overall height of the wall.
- 6.4.8.9.5 Spacing of vertical shear reinforcement shall not exceed the smallest of $l_w/3$, $3h$, and 450 mm, where l_w is the overall length of the wall.

6.4.9 Transfer of moments to columns

- 6.4.9.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, the shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns.
- 6.4.9.2 Except for connections not part of a primary seismic load-resisting system that are restrained on four sides by beams or slabs of approximately equal depth, connections shall have lateral reinforcement not less than that required by Eq. (6.4.9) within the column for a depth not less than that of the deepest connection of framing elements to the columns. See also Sec. 8.1.13.

6.4.10 Provisions for footings and slabs

- 6.4.10.1 The shear strength of footings and slabs in the vicinity of columns, concentrated loads, or reactions is governed by the more severe of the following two conditions:
- 6.4.10.1.1 Beam action where each critical section to be investigated extends in a plane across the entire width. The slab or footing shall be designed in accordance with 6.4.1 through 6.4.3 for beam action.
- 6.4.10.1.2 For two-way action, each of the critical sections to be investigated shall be located so that its perimeter b_o is a minimum but need not approach closer than $d/2$ to:
- Edges or corners of columns, concentrated loads, or reaction areas; and
 - Changes in slab thickness such as edges of capitals, drop panels, or shear caps.
- For two-way action, the slab or footing shall be designed in accordance with 6.4.10.2 through 6.4.10.6.
- 6.4.10.1.3 For square or rectangular columns, concentrated loads, or reaction areas, the critical sections with four straight sides shall be permitted.
- 6.4.10.2 For two-way action, the design of a slab or footing is based on Eq. (6.4.1) and (6.4.2). V_c shall be computed in accordance with 6.4.10.2.1, or 6.4.10.3.1. V_s shall be computed in accordance with 6.4.10.3. For slabs with shearheads, V_n shall be in accordance with 6.4.10.4. Where moment is transferred between a slab and a column, 6.4.10.6 shall apply.

- 6.4.10.2.1 For slabs and footings, V_c shall be the smallest of (a), (b), and (c):

$$a) \quad V_c = 0.17 \left(1 + \frac{2}{\beta} \right) \lambda \sqrt{f'_c} b_o d \quad (6.4.26)$$

where β is the ratio of long side to short side of the column, concentrated load or reaction area;

$$b) \quad V_c = 0.083\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}b_o d \quad (6.4.27)$$

where α_s is 40 for interior columns, 30 for edge columns, 20 for corner columns; and

$$c) \quad V_c = 0.33\lambda\sqrt{f'_c}b_o d \quad (6.4.28)$$

6.4.10.3 Bars or wires and single- or multiple-leg stirrups as shear reinforcement shall be permitted in slabs and footings with d greater than or equal to 150 mm, but not less than 16 times the shear reinforcement bar diameter. Shear reinforcement shall be in accordance with 6.4.10.3.1 through 6.4.10.3.4.

6.4.10.3.1 For computing V_n , Eq. (6.4.2) shall be used and V_c shall not be taken greater than $0.17\lambda\sqrt{f'_c}b_o d$, and V_s shall be calculated in accordance with 6.4.3. In Eq. (6.4.10), A_v shall be taken as the cross-sectional area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section.

6.4.10.3.2 V_n shall not be taken greater than $0.5\sqrt{f'_c}b_o d$.

6.4.10.3.3 The distance from the column face to the first line of stirrup legs that surround the column shall not exceed $d/2$. The spacing between adjacent stirrup legs in the first line of shear reinforcement shall not exceed $2d$ measured in a direction parallel to the column face. The spacing between successive lines of shear reinforcement that surround the column shall not exceed $d/2$ measured in a direction perpendicular to the column face. In a slab-column connection for which the moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section (Fig. 6.4.10.1). Spacing limits defined above are also shown in Fig. 6.4.10.1 for interior column and in Fig. 6.4.10.2 for edge column. At edge columns or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible.

6.4.10.3.4 Slab shear reinforcement shall satisfy the anchorage requirements of 8.2.10 and shall engage the longitudinal flexural reinforcement in the direction being considered.

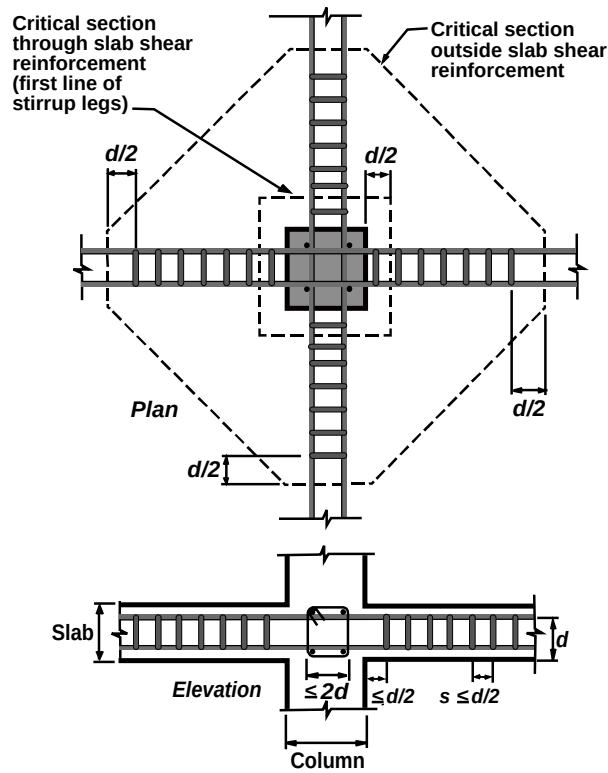


Fig. 6.4.10.1- Arrangement of stirrup shear reinforcement around interior column (6.4.10.3.3).

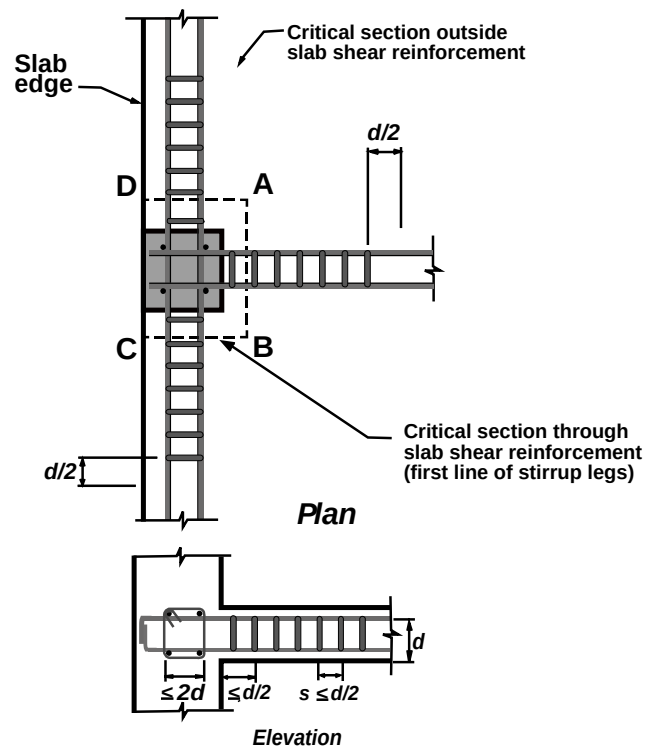


Fig. 6.4.10.1- Arrangement of stirrup shear reinforcement around edge column (6.4.10.3.3).

6.4.10.4 Shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) shall be permitted in slabs. The provisions of 6.4.10.4.1 through 6.4.10.4.9 shall apply where

shear due to gravity load is transferred at interior column supports. Where moment is transferred to columns, 6.4.10.7.3 shall apply.

6.4.10.4.1 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.

6.4.10.4.2 A shearhead shall not be deeper than 70 times the web thickness of the steel shape.

6.4.10.4.3 The ends of each shearhead arm shall be permitted to be cut at angles not less than 30 degrees 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.

6.4.10.4.4 All compression flanges of steel shapes shall be located within $0.3d$ of compression surface of slab.

6.4.10.4.5 The ratio α_v between the flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width $(c_2 + d)$ shall not be less than 0.15.

6.4.10.4.6 Plastic moment strength, M_p , required for each arm of the shearhead shall be computed by

$$M_p = \frac{V_u}{2\phi n} \left[h_v + \alpha_v \left(l_v - \frac{c_1}{2} \right) \right] \quad (6.4.29)$$

where ϕ is for tension-controlled members, n is number of shearhead arms, and l_v is minimum length of each shearhead arm required to comply with requirements of 6.4.10.4.7 and 6.4.10.4.8.

6.4.10.4.7 The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-quarters the distance $[l_v - (c_1/2)]$ from the column face to the end of the shearhead arm. The critical section shall be located so that its perimeter b_o is a minimum, but need not be closer than the perimeter defined in 6.4.10.1.2(a).

6.4.10.4.8 V_n shall not be taken larger than $0.33\sqrt{f'_c}b_o d$ on the critical section defined in 6.4.10.4.7. When shearhead reinforcement is provided, V_n shall not be taken greater than $0.58\sqrt{f'_c}b_o d$ on the critical section defined in 6.4.10.1.2(a).

6.4.10.4.9 Moment resistance M_v contributed to each slab column strip by a shearhead shall not be taken greater than

$$M_v = \frac{\phi \alpha_v V_u}{2n} \left(l_v - \frac{c_1}{2} \right) \quad (6.4.30)$$

where ϕ is for tension-controlled members, n is number of shearhead arms, and l_v is length of each shearhead arm actually provided. However, M_v shall not be taken larger than the smallest of:

- 30 percent of the total factored moment required for each slab column strip;
- The change in column strip moment over the length l_v ;
- M_p computed by Eq. (6.4.29).

6.4.10.4.10 When unbalanced moments are considered, the shearhead must have adequate anchorage to transmit M_p to the column.

6.4.10.5 Headed shear stud reinforcement, placed perpendicular to the plane of a slab or footing, shall be permitted in slabs and footings in accordance with 6.4.10.5.1 through 6.4.10.5.4. The overall height of the shear stud assembly shall not be less than the thickness of the member less the sum of: (1) the concrete cover on the top flexural reinforcement; (2) the concrete cover on the base rail; and (3) one-half the bar diameter of the tension flexural reinforcement.

Where flexural tension reinforcement is at the bottom of the section, as in a footing, the overall height of the shear stud assembly shall not be less than the thickness of the member less the sum of: (1) the concrete cover on the bottom flexural reinforcement; (2) the concrete cover on the head of the stud; and (3) one-half the bar diameter of the bottom flexural reinforcement.

- 6.4.10.5.1 For the critical section defined in 6.4.10.1.2, V_n shall be computed using Eq. (6.4.2), with V_c and V_n not exceeding $0.25\lambda\sqrt{f'_c}b_o d$ and $0.66\sqrt{f'_c}b_o d$, respectively. V_s shall be calculated using Eq. (6.4.10) with A_v equal to the cross-sectional area of all the shear reinforcement on one peripheral line that is approximately parallel to the perimeter of the column section, where s is the spacing of the peripheral lines of headed shear stud reinforcement. $A_v f_{yt} / (b_o s)$ shall not be less than $0.17\sqrt{f'_c}$.
- 6.4.10.5.2 The spacing between the column face and the first peripheral line of shear reinforcement shall not exceed $d/2$. The spacing between peripheral lines of shear reinforcement, measured in a direction perpendicular to any face of the column, shall be constant. For all slabs and footings, the spacing shall be based on the value of the shear stress due to factored shear force and unbalanced moment at the critical section defined in 6.4.10.1.2, and shall not exceed:
- $0.75d$ where maximum shear stresses due to factored loads are less than or equal to $0.5\phi\sqrt{f'_c}$; and
 - $0.5d$ where maximum shear stresses due to factored loads are greater than $0.5\phi\sqrt{f'_c}$.
- 6.4.10.5.3 The spacing between adjacent shear reinforcement elements, measured on the perimeter of the first peripheral line of shear reinforcement, shall not exceed $2d$.
- 6.4.10.5.4 Shear stress due to factored shear force and moment shall not exceed $0.17\phi\lambda\sqrt{f'_c}$ at the critical section located $d/2$ outside the outermost peripheral line of shear reinforcement.

6.4.10.6 Openings in slabs

If 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 Sec. 6.5, the critical slab sections for shear defined in 6.4.10.1.2 and 6.4.10.4.7 shall be modified as follows:

- 6.4.10.6.1 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 column, concentrated load, or reaction area and tangent to the boundaries of the openings shall be considered ineffective (Fig. 6.4.10.3).
- 6.4.10.6.2 For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in 6.4.10.6.1.

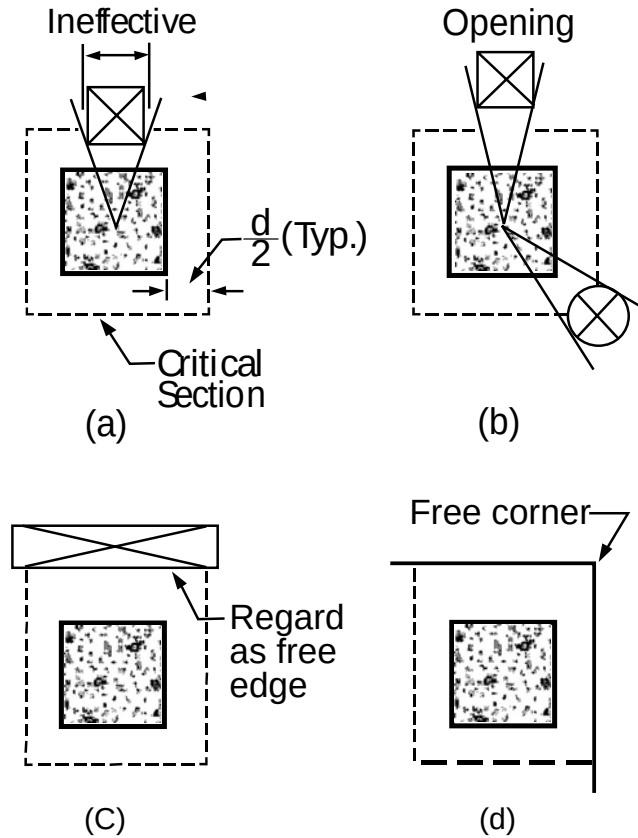


Fig. 6.4.10.3 – Effective perimeter (in dashed lines) to consider effect of openings and free edges (6.4.10.6.1).

6.4.10.7 Transfer of moment in slab-column connections

- 6.4.10.7.1 Where gravity load, wind, earthquake, or other lateral forces cause transfer of unbalanced moment M_u between a slab and column, $\gamma_f M_u$ shall be transferred by flexure in accordance with 6.5.5.3. The remainder of the unbalanced moment, $\gamma_v M_u$, shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in 6.4.10.1.2 where

$$\gamma_v = (1 - \gamma_f) \quad (6.4.31)$$

- 6.4.10.7.2 The shear stress resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly about the centroid of the critical sections defined in 6.4.10.1.2. The maximum shear stress due to V_u and M_u shall not exceed ϕv_n :

- (a) For members without shear reinforcement,

$$\phi V_n = \phi V_c / (b_o d) \quad (6.4.32)$$

where V_c is as defined in 6.4.10.2.1.

(b) For members with shear reinforcement other than shearheads,

$$\phi V_n = \phi(V_c + V_s)/(b_o d) \quad (6.4.33)$$

where V_c and V_s are defined in 6.4.10.3.1. The design shall take into account the variation of shear stress around the column. The shear stress due to factored shear force and moment shall not exceed $\phi(0.17\lambda\sqrt{f'_c})$ at the critical section located $d/2$ outside the outermost line of stirrup legs that surround the column.

The maximum factored shear stress may be obtained from the combined shear stresses on the left and right faces of the column (Fig. 6.4.10.4),

$$v_l = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_l}{J_c}$$

$$v_r = \frac{V_u}{A_c} + \frac{\gamma_v M_u c_r}{J_c}$$

Where, A_c = area of concrete of assumed critical section $= 2d(c_1 + c_2 + 2d)$

c_l, c_r = distances from centroid of critical section to left and right face of section respectively

c_1, c_2 = width and depth of the column

J_c = property of assumed critical section analogous to polar moment of inertia

For an interior column, the quantity J_c is

$$J_c = \frac{2d(c_1 + d)^3}{12} + \frac{2(c_1 + d)d^3}{12} + 2d(c_2 + d)\left(\frac{c_1 + d}{2}\right)^2$$

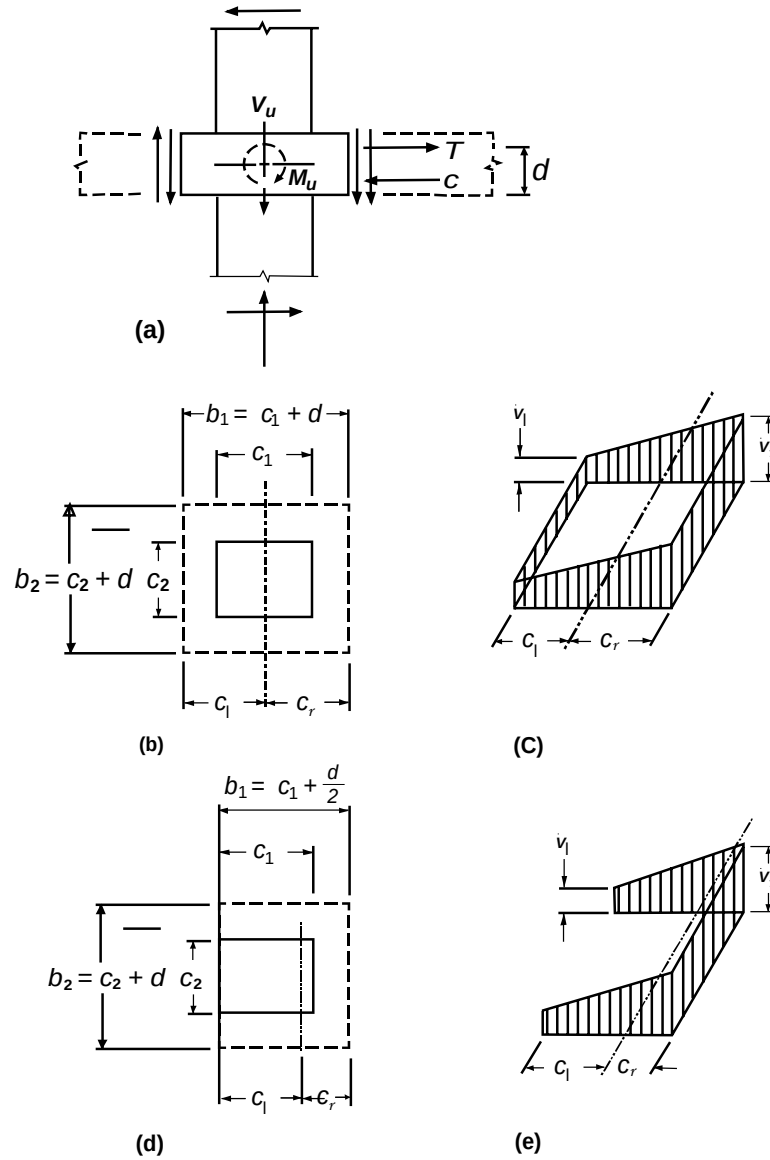


Fig. 6.4.10.4 – Transfer of moment from slab to column: (a) forces resulting from vertical load and unbalanced moment; (b) critical section for an interior column; (c) shear stress distribution for an interior column; (d) critical section for an edge column; (e) shear stress distribution for an edge column (6.4.10.7.2).

- 6.4.10.7.3 When shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) is provided, the sum of the shear stresses due to vertical load acting on the critical section defined by 6.4.10.4.7 and the shear stresses resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in 6.4.10.1.2(a) and 6.4.10.1.3 shall not exceed $\phi 0.33\lambda \sqrt{f'_c}$.

6.5 TWO-WAY SLAB SYSTEMS: FLAT PLATES, FLAT SLABS AND EDGE-SUPPORTED SLABS

6.5.1 Scope

The provisions of this section shall apply to all slabs, solid, ribbed or hollow, spanning in more than one direction, with or without beams between the supports. Flat plate is a term normally attributed to slabs without beams and without drop panels, column capitals, or brackets. On the other hand, slabs without beams, but with drop panels, column capital or brackets are commonly known as flat slabs. While this section covers the requirements for all types of slabs, the provisions of 6.5.8, Alternative Design of Two-way Edge-Supported slabs, may be used as an alternative for slabs supported on all four edges by walls, steel beams or monolithic concrete beams having a total depth not less than 3 times the slab thickness.

- 6.5.1.1 For a slab system supported by columns or walls, dimensions c_1 , c_2 , and l_n shall be based on an effective support area defined by the intersection of the bottom surface of the slab, or of the drop panel or shear cap if present, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and the capital or bracket and are oriented no greater than 45 degrees to the axis of the column.

- 6.5.1.2 Minimum thickness of slabs designed in accordance with Sec. 6.5 shall be as required by 6.2.5.3.

6.5.2 General

- 6.5.2.1 Column strip is a design strip with a width on each side of a column centerline equal to $0.25l_2$ or $0.25l_1$, whichever is less. Column strip includes beams, if any.
- 6.5.2.2 Middle strip is a design strip bounded by two column strips.
- 6.5.2.3 A panel is bounded by column, beam, or wall centerlines on all sides.
- 6.5.2.4 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 (Fig. 6.5.2.1).
- 6.5.2.5 When used to reduce the amount of negative moment reinforcement over a column or minimum required slab thickness, a drop panel shall:
- project below the slab at least one-quarter of the adjacent slab thickness; and
 - extend in each direction from the centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.
- 6.5.2.6 When used to increase the critical condition section for shear at a slab-column joint, a shear cap shall project below the slab and extend a minimum horizontal distance from the face of the column that is equal to the thickness of the projection below the slab soffit.

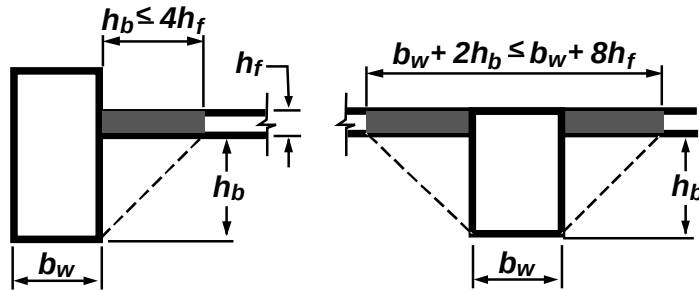
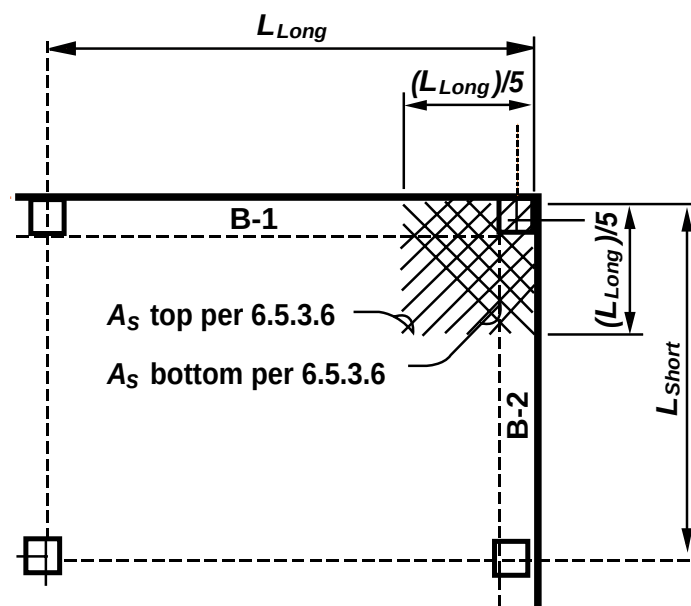


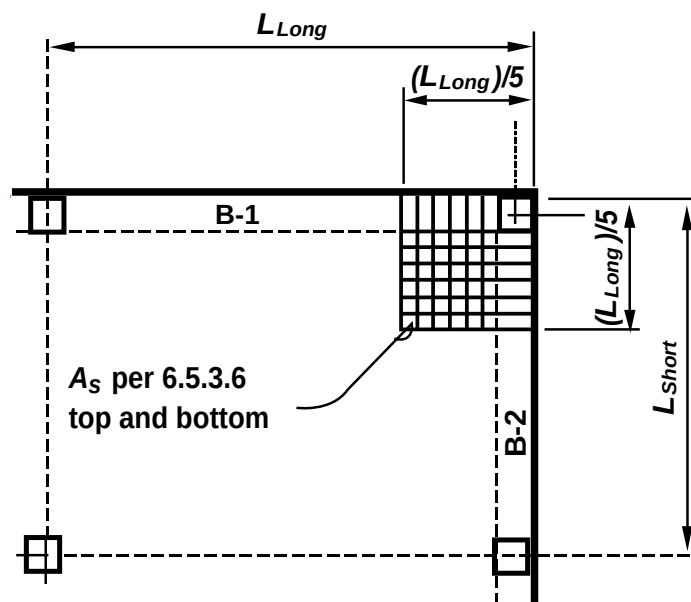
Fig. 6.5.2.1 – Portion of slab to be included with the beam according to 6.5.2.4.

6.5.3 Slab reinforcement

- 6.5.3.1 Area 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 Sec. 8.1.11.2.
- 6.5.3.2 Spacing of reinforcement at critical sections shall not exceed two times the slab thickness, except for portions of slab area of cellular or ribbed construction. In the slab over cellular spaces, reinforcement shall be provided as required by Sec. 8.1.11.
- 6.5.3.3 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.
- 6.5.3.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored in spandrel beams, columns, or walls, and shall be developed at face of support according to provisions of Sec. 8.2.
- 6.5.3.5 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 shall be permitted within the slab.
- 6.5.3.6 At exterior corners of slabs supported by edge walls or where one or more edge beams have a value of α_f greater than 1.0, top and bottom slab reinforcement shall be provided at exterior corners in accordance with 6.5.3.6.1 through 6.5.3.6.4 and as shown in Fig. 6.5.3.1.



Choice-1



Choice-2

Notes:

1. Applies if B-1 or B-2 has $\alpha_f > 1.0$
2. Max. bar spacing $2h$, where h = slab thickness
3. Reinforcement same as maximum +ve reinforcement of the panel

Fig. 6.5.3.1— Corner reinforcement in slabs (6.5.3.6)

- 6.5.3.6.1 Corner reinforcement in both top and bottom of slab shall be sufficient to resist a moment per unit of width equal to the maximum positive moment per unit width in the slab panel.

- 6.5.3.6.2 The moment shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the corner in the bottom of the slab.
- 6.5.3.6.3 Corner reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.
- 6.5.3.6.4 Corner reinforcement shall be placed parallel to the diagonal in the top of the slab and perpendicular to the diagonal in the bottom of the slab. Alternatively, reinforcement shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.
- 6.5.3.7 When a drop panel is used to reduce the amount of negative moment reinforcement over the column of a flat slab, the dimensions of the drop panel shall be in accordance with 6.5.2.5. In computing required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed to be greater than one-quarter the distance from the edge of drop panel to the face of column or column capital.
- 6.5.3.8 Details of reinforcement in slabs without beams
- 6.5.3.8.1 In addition to the other requirements of 6.5.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Fig. 6.5.3.2.
- 6.5.3.8.2 Where adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support as prescribed in Fig. 6.5.3.2 shall be based on requirements of the longer span.

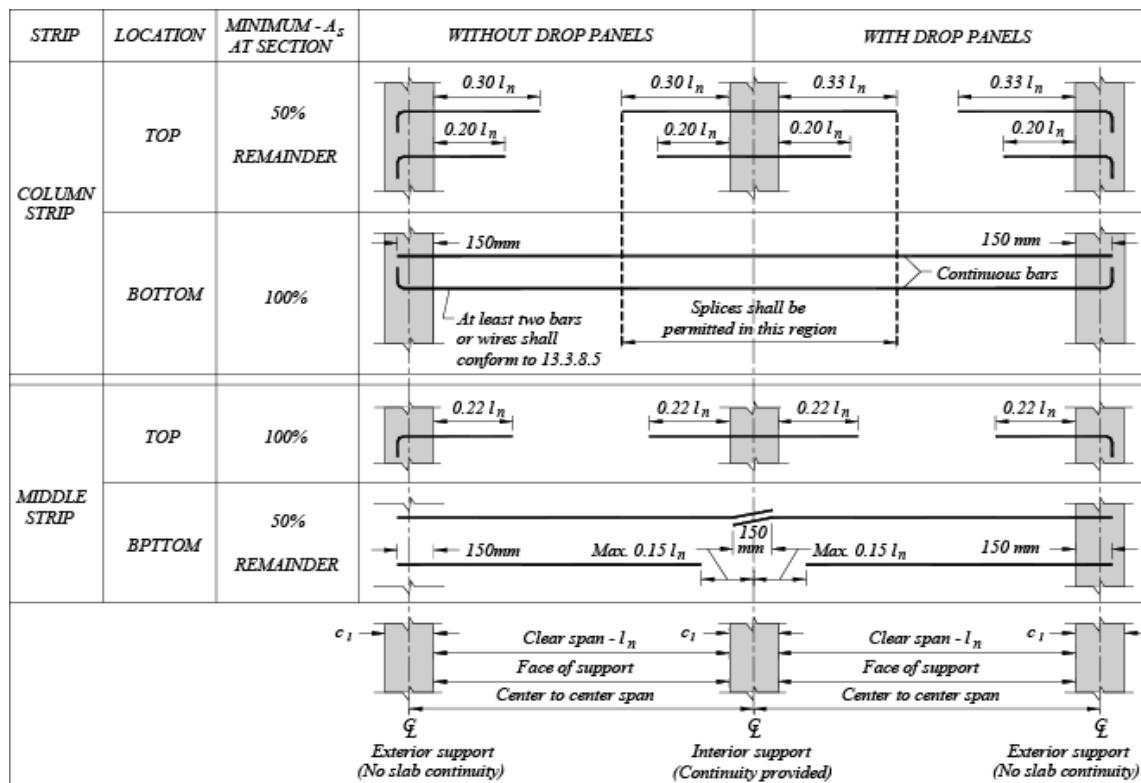


Fig. 6.5.3.2—Minimum extensions for reinforcement in slabs without beams. (See 8.2.8.1 for reinforcement extension into supports)

- 6.5.3.8.3 Bent bars shall be permitted only when depth-span ratio permits use of bends of 45 degrees or less.
- 6.5.3.8.4 In frames where two-way slabs act as primary members resisting lateral loads, lengths of reinforcement shall be determined by analysis but shall not be less than those prescribed in Fig. 6.5.3.2.
- 6.5.3.8.5 All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class B tension splices or with mechanical or welded splices satisfying Sec. 8.2.12.3. Splices shall be located as shown in Fig. 6.5.3.2. At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.
- 6.5.3.8.6 In slabs with shearheads and in lift-slab construction where it is not practical to pass the bottom bars required by 6.5.3.8.5 through the column, at least two bonded bottom bars or wires in each direction shall pass through the shearhead or lifting collar as close to the column as practicable and be continuous or spliced with a Class A splice. At exterior columns, the reinforcement shall be anchored at the shearhead or lifting collar.

6.5.4 Openings in slab systems

- 6.5.4.1 Openings of any size shall be permitted in slab systems if shown by analysis that the design strength is at least equal to the required strength set forth in 6.2.2 and 6.2.3, and that all serviceability conditions, including the limits on deflections, are met.
- 6.5.4.2 As an alternate to analysis as required by 6.5.4.1, openings shall be permitted in slab systems without beams only, in accordance with 6.5.4.2.1 through 6.5.4.2.4.
- 6.5.4.2.1 Openings of any size shall be permitted in the area common to intersecting middle strips, provided total amount of reinforcement required for the panel without the opening is maintained.
- 6.5.4.2.2 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.
- 6.5.4.2.3 In the area common to one column strip and one middle strip, not more than one-quarter of 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.
- 6.5.4.2.4 Shear requirements of 6.4.10.6 shall be satisfied.

6.5.5 Design procedures

- 6.5.5.1 A slab system shall 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 set forth in 6.2.2 and 6.2.3, and that all serviceability conditions, including limits on deflections, are met.
- 6.5.5.1.1 Design of a slab system for gravity loads, including the slab and beams (if any) between supports and supporting columns or walls forming orthogonal frames, by either the Direct Design Method of 6.5.6 or the Equivalent Frame Method of 6.5.7, shall be permitted.

6.5.5.1.2 For lateral loads, analysis of frames shall take into account effects of cracking and reinforcement on stiffness of frame members.

6.5.5.1.3 Combining the results of the gravity load analysis with the results of the lateral load analysis shall be permitted.

6.5.5.2 The slab and beams (if any) between supports shall be proportioned for factored moments prevailing at every section.

6.5.5.3 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 6.5.5.3.2 through 6.5.5.3.4.

6.5.5.3.1 The fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with 6.4.10.7.

6.5.5.3.2 A fraction of the unbalanced moment given by $\gamma_f M_u$ shall be considered to be transferred by flexure within 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, where M_u is the factored moment to be transferred and

$$\gamma_f = \frac{1}{1 + (2/3)\sqrt{b_1/b_2}} \quad (6.5.1)$$

6.5.5.3.3 For slabs with unbalanced moments transferred between the slab and columns, it shall be permitted to increase the value of γ_f given by Eq. (6.5.1) in accordance with the following:

- a) For edge columns with unbalanced moments about an axis parallel to the edge, $\gamma_f = 1.0$ provided that V_u at an edge support does not exceed $0.75\phi V_c$, or at a corner support does not exceed $0.5\phi V_c$.
- b) For unbalanced moments at interior supports, and for edge columns with unbalanced moments about an axis perpendicular to the edge, increase γ_f to as much as 1.25 times the value from Eq. (6.5.1), but not more than $\gamma_f = 1.0$, provided that V_u at the support does not exceed $0.4\phi V_c$. The net tensile strain ϵ_t calculated for the effective slab width defined in 6.5.5.3.2 shall not be less than 0.010.

The value of V_c in items (a) and (b) shall be calculated in accordance with 6.4.10.2.1.

6.5.5.3.4 Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in 6.5.5.3.2.

6.5.5.4 Design for transfer of load from slabs to supporting columns or walls through shear and torsion shall be in accordance with Sec. 6.4.

6.5.6 Direct design method

6.5.6.1 Limitations

Design of slab systems within the limitations of 6.5.6.1.1 through 6.5.6.1.8 by the direct design method shall be permitted.

- 6.5.6.1.1 There shall be a minimum of three continuous spans in each direction.
- 6.5.6.1.2 Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2.
- 6.5.6.1.3 Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span.
- 6.5.6.1.4 Offset of columns by a maximum of 10 percent of the span (in direction of offset) from either axis between centerlines of successive columns shall be permitted.
- 6.5.6.1.5 All loads shall be due to gravity only and uniformly distributed over an entire panel. The unfactored live load shall not exceed two times the unfactored dead load.
- 6.5.6.1.6 For a panel with beams between supports on all sides, Eq. (6.5.2) shall be satisfied for beams in the two perpendicular directions

$$0.2 \leq \frac{\alpha_{f1} \ell_2^2}{\alpha_{f2} \ell_1^2} \leq 5.0 \quad (6.5.2)$$

where

α_{f1} and α_{f2} are calculated in accordance with Eq. (6.5.3).

$$\alpha_f = \frac{E_{cb} I_b}{E_{cs} I_s} \quad (6.5.3)$$

- 6.5.6.1.7 Moment redistribution as permitted by 8.4 shall not be applied for slab systems designed by the direct design method. See 6.5.6.7.
- 6.5.6.1.8 Variations from the limitations of 6.5.6.1 shall be permitted if demonstrated by analysis that requirements of 6.5.5.1 are satisfied.

6.5.6.2 Total factored static moment for a span

- 6.5.6.2.1 Total factored static moment, M_o , for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.
- 6.5.6.2.2 Absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8} \quad (6.5.4)$$

where ℓ_n is length of clear span in direction that moments are being determined.

6.5.6.2.3 Where the transverse span of panels on either side of the centerline of supports varies, l_2 in Eq. (6.5.4) shall be taken as the average of adjacent transverse spans.

6.5.6.2.4 When the span adjacent and parallel to an edge is being considered, the distance from edge to panel centerline shall be substituted for l_2 in Eq. (6.5.4).

6.5.6.2.5 Clear span l_n shall extend from face to face of columns, capitals, brackets, or walls. Value of l_n used in Eq. (6.5.4) shall not be less than $0.65l_1$. Circular or regular polygon-shaped supports shall be treated as square supports with the same area.

6.5.6.3 Negative and positive factored moments

6.5.6.3.1 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.

6.5.6.3.2 In an interior span, total static moment, M_o , shall be distributed as follows:

Negative factored moment	0.65
Positive factored moment	0.35

6.5.6.3.3 In an end span, total factored static moment, M_o , shall be distributed as in Table 6.5.6.1 below:

Table 6.5.6.1 Distribution of Total Factored Static Moment, M_o , in an End Span

	(1)	(2)	(3)	(4)	(5)
	Exterior edge unrestrained	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

6.5.6.3.4 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.

6.5.6.3.5 Edge beams or edges of slab shall be proportioned to resist in torsion their share of exterior negative factored moments.

6.5.6.3.6 The gravity load moment to be transferred between slab and edge column in accordance with 6.5.5.3.1 shall be $0.3M_o$.

6.5.6.4 Factored moments in column strips

6.5.6.4.1 Column strips shall be proportioned to resist the portions in percent of interior negative factored moments as shown in Table 6.5.6.2 below:

Table 6.5.6.2 Portions of Interior Negative Moments to be Resisted by Column Strip

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_{f1}\ell_2/\ell_1) = 0$	75	75	75
$(\alpha_{f1}\ell_2/\ell_1) \geq 1$	90	75	45

Note : Linear interpolations shall be made between values shown.

Interpolation function for % of Moment = $75 + 30 \left(\frac{\alpha_{f1}\ell_2}{\ell_1} \right) \left(1 - \frac{\ell_2}{\ell_1} \right)$

- 6.5.6.4.2 Column strips shall be proportioned to resist the portions in percent of exterior negative factored moments as shown in Table 6.5.6.3:

Table 6.5.6.3 Portions of Exterior Negative Moments to be Resisted by Column Strip

ℓ_2/ℓ_1		0.5	1.0	2.0
$(\alpha_{f1}\ell_2/\ell_1)$	$\beta_t = 0$	100	100	100
$= 0$	$\beta_t \geq 2.5$	075	075	075
$(\alpha_{f1}\ell_2/\ell_1)$	$\beta_t = 0$	100	100	100
≥ 1	$\beta_t \geq 2.5$	090	075	045

Linear interpolations shall be made between values shown, where β_t is calculated in Eq. (6.5.5) and C is calculated in Eq. (6.5.6).

$$\beta_t = \frac{E_{cb}C}{2E_{cs}I_s} \quad (6.5.5)$$

$$C = \sum \left(1 - 0.63 \frac{x}{y} \right) \frac{x^3 y}{3} \quad (6.5.6)$$

The constant C for T- or L-sections shall be permitted to be evaluated by dividing the section into separate rectangular parts, as defined in 6.5.2.4, and summing the values of C for each part.

Interpolation function for % of Moment = $100 - 10\beta_t + 12\beta_t \left(\frac{\alpha_{f1}\ell_2}{\ell_1} \right) \left(1 - \frac{\ell_2}{\ell_1} \right)$

- 6.5.6.4.3 Where supports consist of columns or walls extending for a distance equal to or greater than $(0.75)\ell_2$ used to compute M_o , negative moments shall be considered to be uniformly distributed across ℓ_2 .

- 6.5.6.4.4 Column strips shall be proportioned to resist the portions in percent of positive factored moments shown in Table 6.5.6.4:

Table 6.5.6.4 Portions of Positive Moment to be Resisted by Column Strip

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_{f1}\ell_2/\ell_1) = 0$	60	60	60
$(\alpha_{f1}\ell_2/\ell_1) \geq 1$	90	75	45

Note : Linear interpolations shall be made between values shown.

Interpolation function for % of Moment = $60 + 30 \left(\frac{\alpha_{f1}\ell_2}{\ell_1} \right) \left(1.5 - \frac{\ell_2}{\ell_1} \right)$

6.5.6.4.5 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.

6.5.6.5 Factored moments in beams

6.5.6.5.1 Beams between supports shall be proportioned to resist 85 percent of column strip moments if $\alpha_{f1}l_2/l_1$ is equal to or greater than 1.0.

6.5.6.5.2 For values of $\alpha_{f1}l_2/l_1$ between 1.0 and zero, proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

6.5.6.5.3 In addition to moments calculated for uniform loads according to 6.5.6.2.2, 6.5.6.5.1, and 6.5.6.5.2, beams shall be proportioned to resist all moments caused by concentrated or linear loads applied directly to beams, including weight of projecting beam stem above or below the slab.

6.5.6.6 Factored moments in middle strips

6.5.6.6.1 That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

6.5.6.6.2 Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

6.5.6.6.3 A middle strip adjacent to and parallel with a wall-supported edge shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

6.5.6.7 Modification of factored moments

Modification of negative and positive factored moments by 10 percent shall be permitted provided the total static moment for a panel, M_o , in the direction considered is not less than that required by Eq. (6.5.4).

6.5.6.8 Factored shear in slab systems with beams

6.5.6.8.1 Beams with $\alpha_{f1}l_2/l_1$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas which are bounded by 45-degree lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides (Fig. 6.5.6.1).

6.5.6.8.2 In proportioning beams with $\alpha_{f1}l_2/l_1$ less than 1.0 to resist shear, linear interpolation, assuming beams carry no load at $\alpha_{f1} = 0$, shall be permitted.

6.5.6.8.3 In addition to shears calculated according to 6.5.6.8.1 and 6.5.6.8.2, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

6.5.6.8.4 Computation of slab shear strength on the assumption that load is distributed to supporting beams in accordance with 6.5.6.8.1 or 6.5.6.8.2 shall be permitted. Resistance to total shear occurring on a panel shall be provided.

6.5.6.8.5 Shear strength shall satisfy the requirements of Sec. 6.4.

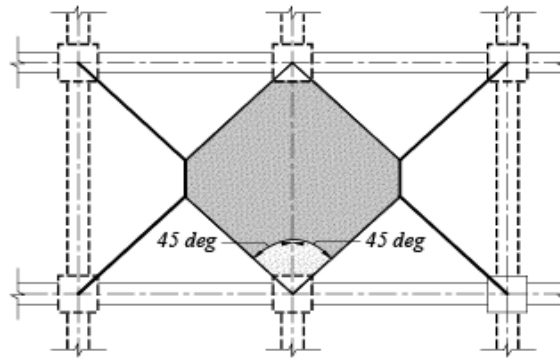


Fig. 6.5.6.1—Tributary area for shear on an interior beam.

6.5.6.9 Factored moments in columns and walls

6.5.6.9.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

6.5.6.9.2 At an interior support, supporting elements above and below the slab shall resist the factored moment specified by Eq. (6.5.7) in direct proportion to their stiffnesses unless a general analysis is made.

$$M_u = 0.07[(q_{Du} + 0.5q_{Lu})\ell_2\ell_n^2 - q'_{Du}\ell'_2(\ell'_n)^2] \quad (6.5.7)$$

Where D'_u , ℓ'_2 , and ℓ'_n refer to shorter span.

6.5.7 Equivalent frame method

6.5.7.1 Design of slab systems by the equivalent frame method shall be based on assumptions given in 6.5.7.2 through 6.5.7.6, and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

6.5.7.1.1 Where metal column capitals are used, it shall be permitted to take account of their contributions to stiffness and resistance to moment and to shear.

6.5.7.1.2 It shall be permitted to neglect the change in length of columns and slabs due to direct stress, and deflections due to shear.

6.5.7.2 Equivalent frame

6.5.7.2.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building (Fig. 6.5.7.1).

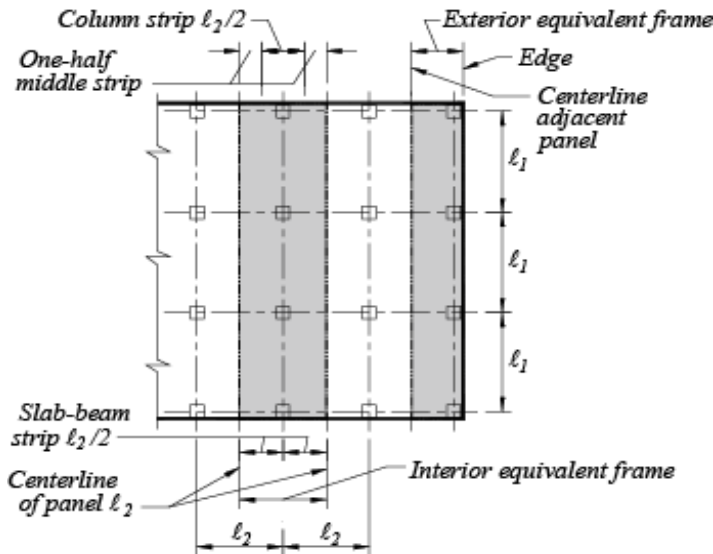


Fig. 6.5.7.1—Definitions of equivalent frame.

- 6.5.7.2.2 Each frame shall consist of a row of columns or supports and slab-beam strips, bounded laterally by the centerline of panel on each side of the centerline of columns or supports.
- 6.5.7.2.3 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members (see 6.5.7.5) 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.
- 6.5.7.2.4 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of adjacent panel.
- 6.5.7.2.5 Analysis of each equivalent frame in its entirety shall be permitted. Alternatively, for gravity loading, a separate analysis of each floor or roof with far ends of columns considered fixed shall be permitted.
- 6.5.7.2.6 Where slab-beams are analyzed separately, determination of moment at a given support assuming that the slab-beam is fixed at any support two panels distant therefrom, shall be permitted, provided the slab continues beyond that point.
- 6.5.7.3 **Slab-beams**
 - 6.5.7.3.1 Determination of the moment of inertia of slab-beams at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.
 - 6.5.7.3.2 Variation in moment of inertia along axis of slab-beams shall be taken into account.
 - 6.5.7.3.3 Moment of inertia of slab-beams from center 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.

6.5.7.4 Columns

6.5.7.4.1 Determination of the moment of inertia of columns at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

6.5.7.4.2 Variation in moment of inertia along axis of columns shall be taken into account (Fig. 6.5.7.2).

6.5.7.4.3 Moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed to be infinite.

6.5.7.5 Torsional members

6.5.7.5.1 Torsional members (see 6.5.7.2.3) shall be assumed to have a constant cross section throughout their length consisting of the largest of (a), (b), and (c):

- 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;
- 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;
- The transverse beam as defined in 6.5.2.4.

6.5.7.5.2 Where beams frame into columns in the direction of the span for which moments are being determined, the torsional stiffness shall be multiplied by the ratio of the moment of inertia of the slab with such a beam to the moment of inertia of the slab without such a beam.

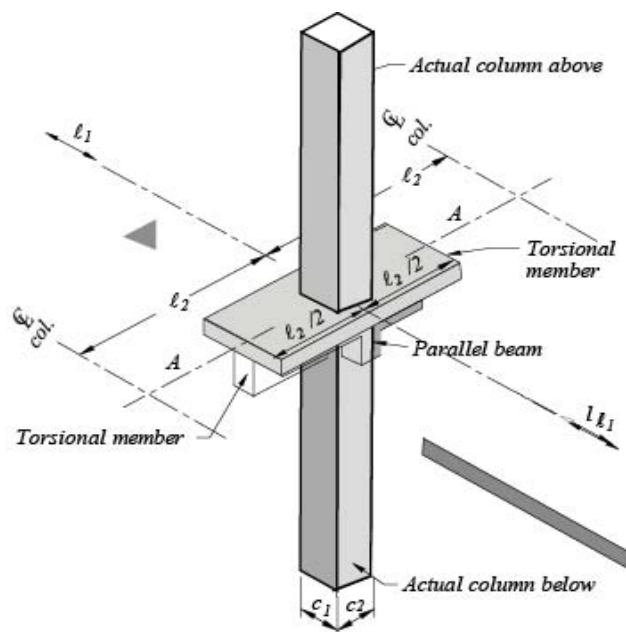


Fig. 6.5.7.2—Equivalent column (column plus torsional members).

6.5.7.5.3 Stiffness K_t of the torsional members shall be calculated by the following expression:

$$K_t = \sum \frac{9E_{cs}C}{\ell_2(1-C_2/\ell_2)^3} \quad (6.5.8)$$

where c_2 and ℓ_2 relate to the transverse span on each side of column.

6.5.7.6 Arrangement of live load

6.5.7.6.1 When the loading pattern is known, the equivalent frame shall be analyzed for that load.

6.5.7.6.2 When the unfactored live load is variable but does not exceed three-quarters of the unfactored dead load, or the nature of live load is such that all panels will be loaded

simultaneously, it shall be permitted to assume that maximum factored moments occur at all sections with full factored live load on entire slab system.

6.5.7.6.3 For loading conditions other than those defined in 6.5.7.6.2, it shall be permitted to assume that maximum positive factored moment near midspan of a panel occurs with three-quarters of the full factored live load on the panel and on alternate panels; and it shall be permitted to assume that maximum negative factored moment in the slab at a support occurs with three-quarters of the full factored live load on adjacent panels only.

6.5.7.6.4 Factored moments shall be taken not less than those occurring with full factored live load on all panels.

6.5.7.7 **Factored moments**

6.5.7.7.1 At interior supports, the critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not farther away than $0.175l_1$ from the center of a column.

6.5.7.7.2 At exterior supports with brackets or capitals, the 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.

6.5.7.7.3 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.

6.5.7.7.4 Where slab systems within limitations of 6.5.6.1 are analyzed by the equivalent frame method, it shall be permitted to reduce the resulting computed moments 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. (6.5.4).

6.5.7.7.5 Distribution of moments at critical sections across the slab-beam strip of each frame to column strips, beams, and middle strips as provided in 6.5.6.4, 6.5.6.5, and 6.5.6.6 shall be permitted if the requirement of 6.5.6.1.6 is satisfied.

6.5.8 **ALTERNATIVE DESIGN OF TWO-WAY EDGE-SUPPORTED SLABS**

6.5.8.1 **Notation**

The notations provided below are applicable for Section 6.5.8 only. Also see Section 6.1.1.1 for other notations.

C_a, C_b	=	Moment coefficients
l_a	=	Length of clear span in short direction
l_b	=	Length of clear span in long direction
M_a	=	Moment in the short direction
M_b	=	Moment in the long direction
w	=	Uniform load
l_1	=	Length of clear span in direction that moment are being determined
l_2	=	Length of clear span transverse to l_1
α_f	=	Ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centre line of adjacent panels (if any) on each side of the beam. See 6.4.2.1
α_{f1}	=	α_f in the direction of l_1 .

6.5.8.2 Scope and Limitations

6.5.8.2.1 The provisions of this section may be used as alternative to those of 6.5.1 through 6.5.7 for two-way slabs supported on all four edges by walls, steel beams or monolithic concrete beams having a total depth not less than 3 times the slab thickness.

6.5.8.2.2 Panels shall be rectangular with a ratio of longer to shorter span centre to centre of supports not greater than 2.

6.5.8.2.3 The value of $(\alpha_{f1}\ell_2/\ell_1)$ shall be greater than or equal to 1.

6.5.8.3 Analysis by the Coefficient Method

6.5.8.3.1 The negative moments and dead load and live load positive moments in the two directions shall be computed from Tables 6.5.8.1, 6.5.8.2 and 6.5.8.3 respectively. Shear in the slab and loads on the supporting beams shall be computed from Table 6.5.8.4.

6.5.8.4 Shear on Supporting Beam

The shear requirements provided in 6.5.6.8 shall be satisfied.

6.5.8.5 Deflection

Thickness of slabs supported on walls or stiff beams on all sides shall satisfy the requirements of 6.2.5.3.

Table 6.5.8.1 Coefficients for Negative Moments in Slabs [†]

$$M_{a,neg} = C_{a,neg} w l_a^2$$

$$M_{b,neg} = C_{b,neg} w l_b^2 \text{ where } w = \text{total uniform dead plus live load per unit area}$$

Ratio $m = \frac{l_a}{l_b}$	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
									
$C_{a,neg}$ 1.00		0.045		0.050	0.075	0.071		0.033	0.061
$C_{b,neg}$		0.045	0.076	0.050			0.071	0.061	0.033
$C_{a,neg}$ 0.95		0.050		0.055	0.079	0.075		0.038	0.065
$C_{b,neg}$		0.041	0.072	0.045			0.067	0.056	0.029
$C_{a,neg}$ 0.90		0.055		0.060	0.080	0.079		0.043	0.068
$C_{b,neg}$		0.037	0.070	0.040			0.062	0.052	0.025
$C_{a,neg}$ 0.85		0.060		0.066	0.082	0.083		0.049	0.072
$C_{b,neg}$		0.031	0.065	0.034			0.057	0.046	0.021
$C_{a,neg}$ 0.80		0.065		0.071	0.083	0.086		0.055	0.075
$C_{b,neg}$		0.027	0.061	0.029			0.051	0.041	0.017
$C_{a,neg}$ 0.75		0.069		0.076	0.085	0.088		0.061	0.078
$C_{b,neg}$		0.022	0.056	0.024			0.044	0.036	0.014

$C_{a,neg}$ 0.70	0.074		0.081	0.086	0.091		0.068	0.081
$C_{b,neg}$	0.017	0.050	0.019			0.038	0.029	0.011
$C_{a,neg}$ 0.65	0.077		0.085	0.087	0.093		0.074	0.083
$C_{b,neg}$	0.014	0.043	0.015			0.031	0.024	0.008
$C_{a,neg}$ 0.60	0.081		0.089	0.088	0.095		0.080	0.085
$C_{b,neg}$	0.010	0.035	0.011			0.024	0.018	0.006
$C_{a,neg}$ 0.55	0.084		0.092	0.089	0.096		0.085	0.086
$C_{b,neg}$	0.007	0.028	0.008			0.019	0.014	0.005
$C_{a,neg}$ 0.50	0.086		0.094	0.090	0.097		0.089	0.088
$C_{b,neg}$	0.006	0.022	0.006			0.014	0.010	0.003

† A crosshatched edge indicates that the slab continues across, or is fixed at the support;
an unmarked edge indicates a support at which torsional resistance is negligible.

Table 6.5.8.2 Coefficients for Dead Load Positive Moments in Slabs †

$$M_{a,pos,dl} = C_{a,dl} w l_a^2$$

$$M_{b,pos,dl} = C_{b,dl} w l_b^2$$

where w = uniform dead load per unit area

Ratio $m = \frac{l_a}{l_b}$	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
									
$C_{a,dl}$ 1.00	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
$C_{b,dl}$	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
$C_{a,dl}$ 0.95	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
$C_{b,dl}$	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
$C_{a,dl}$ 0.90	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
$C_{b,dl}$	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
$C_{a,dl}$ 0.85	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
$C_{b,dl}$	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
$C_{a,dl}$ 0.80	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
$C_{b,dl}$	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
$C_{a,dl}$ 0.75	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
$C_{b,dl}$	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
$C_{a,dl}$ 0.70	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
$C_{b,dl}$	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006

$C_{a,dl}$ 0.65	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.034
$C_{b,dl}$	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
$C_{a,dl}$ 0.60	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
$C_{b,dl}$	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
$C_{a,dl}$ 0.55	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
$C_{b,dl}$	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
$C_{a,dl}$ 0.50	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
$C_{b,dl}$	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

† A crosshatched edge indicates that the slab continues across, or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 6.5.8.3 Coefficients for Live Load Positive Moments in Slabs †

$$M_{a,pos,ll} = C_{a,ll} w l_a^2$$

$$M_{b,pos,ll} = C_{b,ll} w l_b^2 \quad \text{where } w = \text{uniform live load per unit area}$$

Ratio $m = \frac{l_a}{l_b}$	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
									
$C_{a,ll}$ 1.00	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
$C_{b,ll}$	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
$C_{a,ll}$ 0.95	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
$C_{b,ll}$	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
$C_{a,ll}$ 0.90	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
$C_{b,ll}$	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
$C_{a,ll}$ 0.85	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
$C_{b,ll}$	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
$C_{a,ll}$ 0.80	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
$C_{b,ll}$	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
$C_{a,ll}$ 0.75	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
$C_{b,ll}$	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
$C_{a,ll}$ 0.70	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
$C_{b,ll}$	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
$C_{a,ll}$ 0.65	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
$C_{b,ll}$	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009

$C_{a,II}$ 0.60	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
$C_{b,II}$	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
$C_{a,II}$ 0.55	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
$C_{b,II}$	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
$C_{a,II}$ 0.50	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
$C_{b,II}$	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004

† A crosshatched edge indicates that the slab continues across, or is fixed at the support;
an unmarked edge indicates a support at which torsional resistance is negligible.

6.5.8.6 Reinforcement

- 6.5.8.6.1 Area of reinforcement in each direction shall be determined from moments at critical sections but shall not be less than that required by 8.1.11.
- 6.5.8.6.2 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 shall be provided as required by 8.1.11.
- 6.5.8.6.3 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.

Table 6.5.8.4 Ratio of Total Load w in l_a and l_b Directions (w_a and w_b) for Shear in Slab and Load on Supports †

Ratio $m = \frac{l_a}{l_b}$	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
w_a 1.00									
w_b	0.50	0.50	0.17	0.50	0.83	0.71	0.29	0.33	0.67
w_a 0.95	0.55	0.55	0.20	0.55	0.86	0.75	0.33	0.38	0.71
w_b	0.45	0.45	0.80	0.45	0.14	0.25	0.67	0.62	0.29
w_a 0.90	0.60	0.60	0.23	0.60	0.88	0.79	0.38	0.43	0.75
w_b	0.40	0.40	0.77	0.40	0.12	0.21	0.62	0.57	0.25
w_a 0.85	0.66	0.66	0.28	0.66	0.90	0.83	0.43	0.49	0.79
w_b	0.34	0.34	0.72	0.34	0.10	0.17	0.57	0.51	0.21

w_a 0.80	0.71	0.71	0.33	0.71	0.92	0.86	0.49	0.55	0.83
w_b	0.29	0.29	0.67	0.29	0.08	0.14	0.51	0.45	0.17
w_a 0.75	0.76	0.76	0.39	0.76	0.94	0.88	0.56	0.61	0.86
w_b	0.24	0.24	0.61	0.24	0.06	0.12	0.44	0.39	0.14
w_a 0.70	0.81	0.81	0.45	0.81	0.95	0.91	0.62	0.68	0.89
w_b	0.19	0.19	0.55	0.19	0.05	0.09	0.38	0.32	0.11
w_a 0.65	0.85	0.85	0.53	0.85	0.96	0.93	0.69	0.74	0.92
w_b	0.15	0.15	0.47	0.15	0.04	0.07	0.31	0.26	0.08
w_a 0.60	0.89	0.89	0.61	0.89	0.97	0.95	0.76	0.80	0.94
w_b	0.11	0.11	0.39	0.11	0.03	0.05	0.24	0.20	0.06
w_a 0.55	0.92	0.92	0.69	0.92	0.98	0.96	0.81	0.85	0.95
w_b	0.08	0.08	0.31	0.08	0.02	0.04	0.19	0.15	0.05
w_a 0.50	0.94	0.94	0.76	0.94	0.99	0.97	0.86	0.89	0.97
w_b	0.06	0.06	0.24	0.06	0.01	0.03	0.14	0.11	0.03

† A crosshatched edge indicates that the slab continues across, or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

6.5.8.6.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, and shall be developed at face of support according to provisions of Sec 8.2.

6.5.8.6.5 Corner Reinforcement

- Corner reinforcement shall be provided at exterior corners in both bottom and top of the slab, for a distance in each direction from the corner equal to one-fifth the longer span of the corner panel as per provisions of 6.5.3.6.

6.5.9 RIBBED AND HOLLOW SLABS

6.5.9.1 General

The provisions of this section shall apply to slabs constructed in one of the ways described below:

- As a series of concrete ribs with topping cast on forms which may be removed after the concrete has set;
- As a series of concrete ribs between precast blocks which remain part of the completed structure; the top of the ribs may be connected by a topping of concrete of the same strength as that used in the ribs; and
- Slabs with a continuous top and bottom face but containing voids of rectangular, oval or other shape.

6.5.9.2 Analysis and Design

Any method of analysis which satisfies equilibrium and compatibility requirements may be used for ribbed and hollow slabs. Approximate moments and shears in continuous one-way ribbed or hollow slabs may be obtained from 6.1.4.3. For two-way slabs, the unified design approach specified in Sec 6.5 Flat Plates, Flat Slabs and Edge-supported Slabs, shall be used.

6.5.9.3 Shear

6.5.9.3.1 When burnt tile or concrete tile fillers of material having the same strength as the specified strength of concrete in the ribbed and hollow slabs are used permanently, it is permitted to include the vertical shells of fillers in contact with the ribs for shear and negative-moment strength computations, provided adequate bond between the two can be ensured.

6.5.9.3.2 Adequate shear strength of slabs shall be provided in accordance with the requirements of 6.4.10. For one-way ribbed and hollow slab construction, contribution of concrete to shear strength V_c is permitted to be 10 percent more than that specified in 6.4.2. It is permitted to increase shear strength using shear reinforcement or by widening the ends of ribs.

6.5.9.4 Deflection

The recommendations for deflection with respect to solid slabs may be applied to ribbed and hollow slab. Total depth of one-way ribbed and hollow slabs shall not be less than those required by Table 6.2.5.1 in 6.2.5.2. For other slabs the provisions of 6.2.5.3 shall apply.

6.5.9.5 Size and Position of Ribs

In-situ-ribs shall be not less than 100 mm wide. They shall be spaced at centres not greater than 750 mm apart and their depth, excluding any topping, shall be not more than three and half times their width. Ribs shall be formed along each edge parallel to the span of one-way slabs.

6.5.9.6 Reinforcement

The recommendations given in 8.1.6 regarding maximum distance between bars apply to areas of solid concrete in this form of construction. The curtailment, anchorage and cover to reinforcement shall be as specified below:

- a) At least 50 per cent of the total main reinforcement shall be carried through the bottom on to the bearing and anchored in accordance with 8.2.8.
- b) Where a slab, which is continuous over supports, has been designed as simply supported, reinforcement shall be provided over the support to control cracking. This reinforcement shall have a cross-sectional area of not less than one quarter of that required in the middle of the adjoining spans and shall extend at least one-tenth of the clear span into adjoining spans.
- c) In slabs with permanent blocks, the side cover to the reinforcement shall not be less than 10 mm. In all other cases, cover shall be provided according to 8.1.7.

6.6 WALLS

6.6.1 Scope

6.6.1.1 Provisions of Sec. 6.6 shall apply for design of walls subjected to axial load, with or without flexure.

6.6.1.2 Cantilever retaining walls are designed according to flexural design provisions of Sec. 6.3 with minimum horizontal reinforcement according to 6.6.3.3.

6.6.2 General

- 6.6.2.1 Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.
- 6.6.2.2 Walls subject to axial loads shall be designed in accordance with 6.6.2, 6.6.3, and either 6.6.4, 6.6.5, or 6.6.8.
- 6.6.2.3 Design for shear shall be in accordance with 6.4.8.
- 6.6.2.4 Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for each concentrated load shall not exceed the smaller of the center-to-center distance between loads, and the bearing width plus four times the wall thickness.
- 6.6.2.5 Compression members built integrally with walls shall conform to 6.3.8.2.
- 6.6.2.6 Walls shall be anchored to intersecting elements, such as floors and roofs; or to columns pilasters, buttresses, of intersecting walls; and to footings.
- 6.6.2.7 Quantity of reinforcement and limits of thickness required by 6.6.3 and 6.6.5 shall be permitted to be waived where structural analysis shows adequate strength and stability.
- 6.6.2.8 Transfer of force to footing at base of wall shall be in accordance with 6.8.8.
- 6.6.3 Minimum reinforcement**
- 6.6.3.1 Minimum vertical and horizontal reinforcement shall be in accordance with 6.6.3.2 and 6.6.3.3 unless a greater amount is required for shear by 6.4.8.8 and 6.4.8.9.
- 6.6.3.2 Minimum ratio of vertical reinforcement area to gross concrete area, ρ_t , shall be:
- 0.0012 for deformed bars not larger than $\Phi 16$ mm with f_y not less than 420 MPa; or
 - 0.0015 for other deformed bars; or
 - 0.0012 for welded wire reinforcement not larger than MW200 or MD200.
- 6.6.3.3 Minimum ratio of horizontal reinforcement area to gross concrete area, ρ_t , shall be:
- 0.0020 for deformed bars not larger than $\Phi 16$ mm with f_y not less than 420 MPa; or
 - 0.0025 for other deformed bars; or
 - 0.0020 for welded wire reinforcement not larger than MW200 or MD200.
- 6.6.3.4 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:
- One layer consisting of not less than one-half and not more than two-thirds of total reinforcement required for each direction shall be placed not less than 50 mm nor more than one-third the thickness of wall from the exterior surface;
 - 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 the interior surface.
- 6.6.3.5 Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, nor farther apart than 450 mm.
- 6.6.3.6 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.
- 6.6.3.7 In addition to the minimum reinforcement required by 6.6.3.1, not less than two $\Phi 16$ mm bars in walls having two layers of reinforcement in both directions and one $\Phi 16$ mm bar in

walls having a single layer of reinforcement in both directions shall be provided around window, door, and similar sized openings. Such bars shall be anchored to develop f_y in tension at the corners of the openings.

6.6.4 Design of walls as compression members

Except as provided in 6.6.5, walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with provisions of 6.3.2, 6.3.3, 6.3.10, 6.3.11, 6.3.14, 6.6.2, and 6.6.3.

6.6.5 Empirical method of design

6.6.5.1 Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of 6.6.5 if the resultant of all factored loads is located within the middle third of the overall thickness of the wall and all limits of 6.6.2, 6.6.3, and 6.6.5 are satisfied.

6.6.5.2 Design axial strength ϕP_n of a wall satisfying limitations of 6.6.5.1 shall be computed by Eq. (6.6.1) unless designed in accordance with 6.6.4.

$$\phi P_n = 0.55 \phi f'_c A_g \left[1 - \left(\frac{K l_c}{32 h} \right)^2 \right] \quad (6.6.1)$$

where ϕ shall correspond to compression-controlled sections in accordance with 6.2.3.2.2 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, bottom, or	0.8
b) Unrestrained against rotation at both ends	1.0

For walls not braced against lateral translation	2.0

6.6.5.3 Minimum thickness of walls designed by empirical design method

6.6.5.3.1 Thickness of bearing walls shall not be less than 1/25 the supported height or length, whichever is shorter, nor less than 100 mm.

6.6.5.3.2 Thickness of exterior basement walls and foundation walls shall not be less than 190 mm.

6.6.6 Nonbearing walls

6.6.6.1 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.

6.6.7 Walls as grade beams

6.6.7.1 Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of 6.3.2 through 6.3.7. Design for shear shall be in accordance with provisions of Sec. 6.4.

6.6.7.2 Portions of grade beam walls exposed above grade shall also meet requirements of 6.6.3.

6.6.8 Alternative design of slender walls

6.6.8.1 When flexural tension controls the out-of-plane design of a wall, the requirements of 6.6.8 are considered to satisfy 6.3.10.

6.6.8.2 Walls designed by the provisions of 6.6.8 shall satisfy 6.6.8.2.1 through 6.6.8.2.6.

6.6.8.2.1 The wall panel shall be designed as a simply supported, axially loaded member subjected to an out-of-plane uniform lateral load, with maximum moments and deflections occurring at midspan.

6.6.8.2.2 The cross section shall be constant over the height of the panel.

6.6.8.2.3 The wall shall be tension-controlled.

6.6.8.2.4 Reinforcement shall provide a design Strength

$$\phi M_n \geq M_{cr} \quad (6.6.2)$$

where M_{cr} shall be obtained using the modulus of rupture, f_r , given by Eq. (6.2.3).

6.6.8.2.5 Concentrated gravity loads applied to the wall above the design flexural section shall be assumed to be distributed over a width:

- Equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section; but
- Not greater than the spacing of the concentrated loads; and
- Not extending beyond the edges of the wall panel.

6.6.8.2.6 Vertical stress P_u/A_g at the midheight section shall not exceed $0.06f'_c$.

6.6.8.3 Design moment strength ϕM_n for combined flexure and axial loads at midheight shall be

$$\phi M_n \geq M_u \quad (6.6.3)$$

where

$$M_u = M_{ua} + P_u \Delta_u \quad (6.6.4)$$

M_{ua} is the maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including $P\Delta$ effects, and Δ_u is

$$\Delta_u = \frac{5M_{ua}l_c^2}{(0.75)48E_cI_{cr}} \quad (6.6.5)$$

M_u shall be obtained by iteration of deflections, or by Eq. (6.6.6).

$$M_u = \frac{M_{ua}}{1 - \frac{5P_u l_c^2}{(0.75)48E_cI_{cr}}} \quad (6.6.6)$$

where

$$I_{cr} = \frac{E_s}{E_c} \left(A_s + \frac{P_u}{f_y} \frac{h}{2d} \right) (d - c)^2 + \frac{l_w c^3}{3} \quad (6.6.7)$$

and the value of E_s/E_c shall not be taken less than 6.

6.6.8.4 Maximum out-of-plane deflection, Δ_s , due to service loads, including $P\Delta$ effects, shall not exceed $l_c/150$.

If M_a , maximum moment at midheight of wall due to service lateral and eccentric vertical loads, including $P\Delta$ effects, exceeds $(2/3)M_{cr}$, Δ_s shall be calculated by Eq. (6.6.8)

$$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})} (\Delta_n - (2/3)\Delta_{cr}) \quad (6.6.8)$$

If M_a does not exceed $(2/3)M_{cr}$, Δ_s shall be calculated by Eq. (6.6.9)

$$\Delta_s = \left(\frac{M_a}{M_{cr}} \right) \Delta_{cr} \quad (6.6.9)$$

where

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g} \quad (6.6.10)$$

$$\Delta_n = \frac{5M_n l_c^2}{48E_cI_{cr}} \quad (6.6.11)$$

I_{cr} shall be calculated by Eq. (6.6.7), and M_a shall be obtained by iteration of deflections.

6.7 Stairs:

Stairs are the structural elements designed to connect different floors. The stairs shall be designed to meet the minimum load requirements. The flight arrangements, configuration and support conditions (Figure 6.7.1.1) shall govern the design procedure to follow.

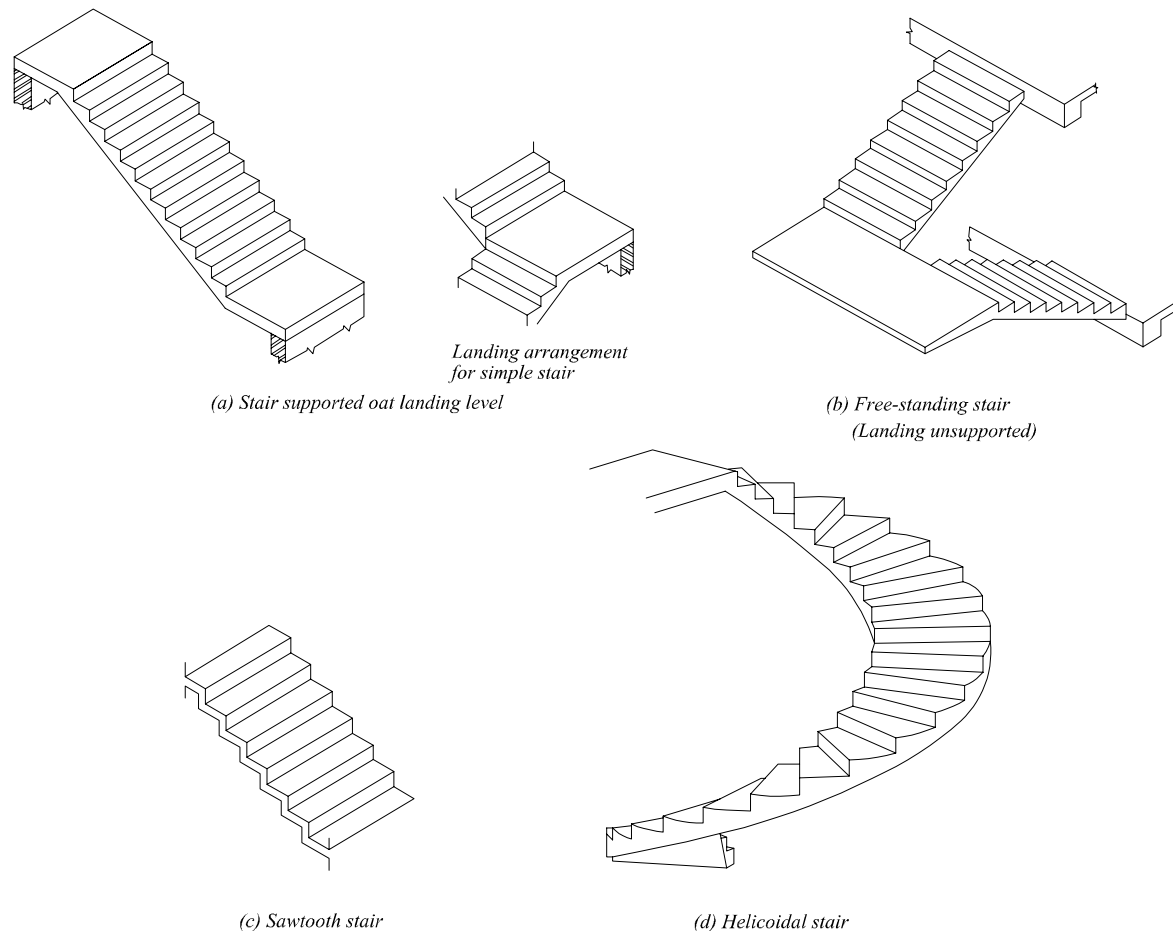


Figure 6.7.1.1: Different forms of stairs and landing arrangements

6.7.1 Stairs supported at landing level

6.7.1.1 Effective span

The effective span of stairs without stringer beams shall be taken as the following horizontal distances:

- Centre to centre distance of beams, where supported at top and bottom risers by beams spanning parallel with the risers,
- Where supported at the edge of a landing slab, which spans parallel with the risers, (Figure 6.7.1.2a) a distance equal to the going of the stairs plus at each end either half the width of the landing or 1.0m whichever is smaller. The going shall be measured horizontally.
- Where the landing spans in the same direction of the stairs (Figure 6.7.1.2b), the span shall be the distance centre to centre of the supporting beams or walls.
- Where the landing slabs, running at right angle to the direction of the flight, supported by walls or beams on three sides (Figure 6.7.1.2c), the effective span shall be going of the stair measured horizontally. Both positive and negative moments along the direction of the flight shall be calculated as $wl^2/8$, where w is the intensity of the total dead and live load per unit area on a horizontal plane.

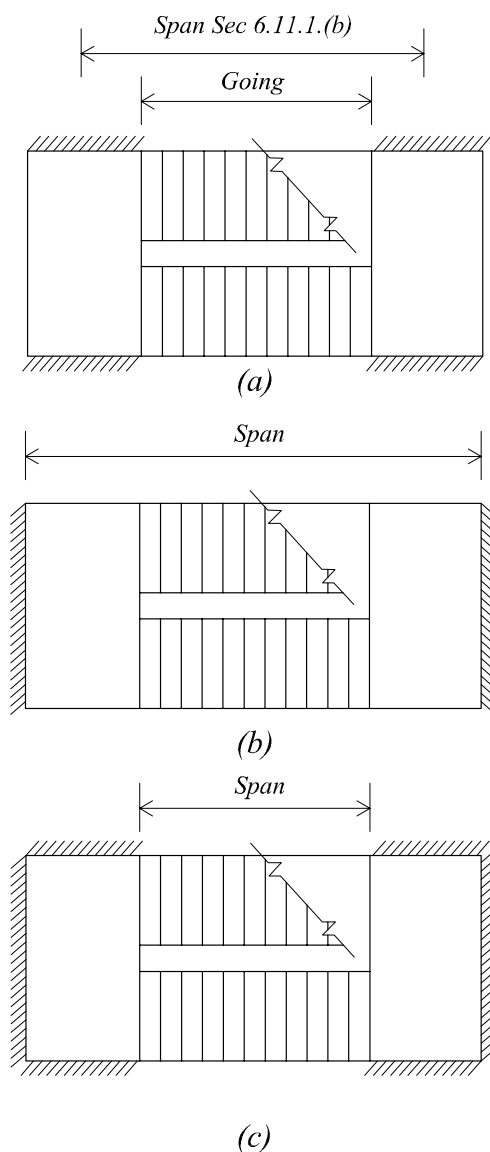


Figure 6.7.1.2: Effective Span for Stairs Supported at Each End by Landings

6.7.1.2 Loading

Staircases shall be designed to support the design ultimate load according to the load combinations specified in Chapter 2, loads.

6.7.1.3 Distribution of loading

6.7.1.3.1 Where flights or landing are embedded at least 110 mm into walls and are designed to span in the direction of the flight, a 150 mm strip may be deducted from the loaded area and the effective breadth of the section may be increased by 75 mm for the purpose of design (Figure 6.7.1.3)

6.7.1.3.2 In the case of stairs with open wells, where spans cross at right angles, the load on areas common to any two such spans may be taken as one half in each direction as shown in Figure 6.7.1.4.

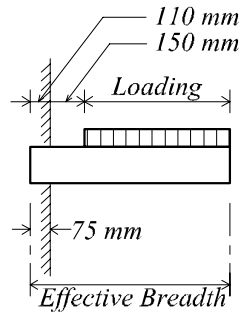


Figure 6.7.1.3: Loading on stairs Built in a wall

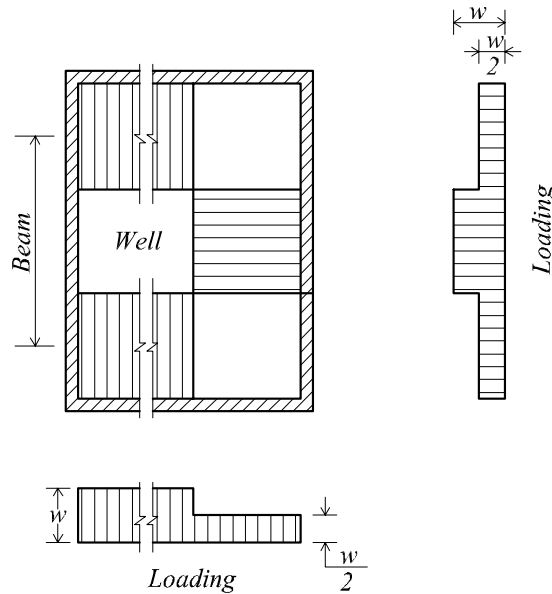


Figure 6.7.1.4: Loading of Stairs with open wells

6.7.1.4 Depth of section

The depth of the section shall be taken as the minimum thickness perpendicular to the soffit of the staircase.

6.7.1.5 Design

6.7.1.5.1 Strength, Deflection and Crack Control

The recommendations given in Sec 6.1 & 6.2 for beams and one-way slabs shall apply, except for the span/depth ratio of staircases without stringer beam where the provision of 6.7.1.5.2 below shall apply.

- 6.7.1.5.2 Permissible span/effective depth ratio for staircase without stringer beams. Provided the stair flight occupies at least 60% of the span, the ratio calculated in accordance with 6.2.5.2 shall be increased by 15%.

6.7.2 Free standing stair (landing unsupported)

6.7.2.1 Effective Span

The effective span for flights and landings of free standing stairs are given in Figure 6.7.2.1. In taking these distances, the spans shall be measured as the centre to centre distance between the stinger beams. In stairs without stinger beams, span shall be measured in between the edges.

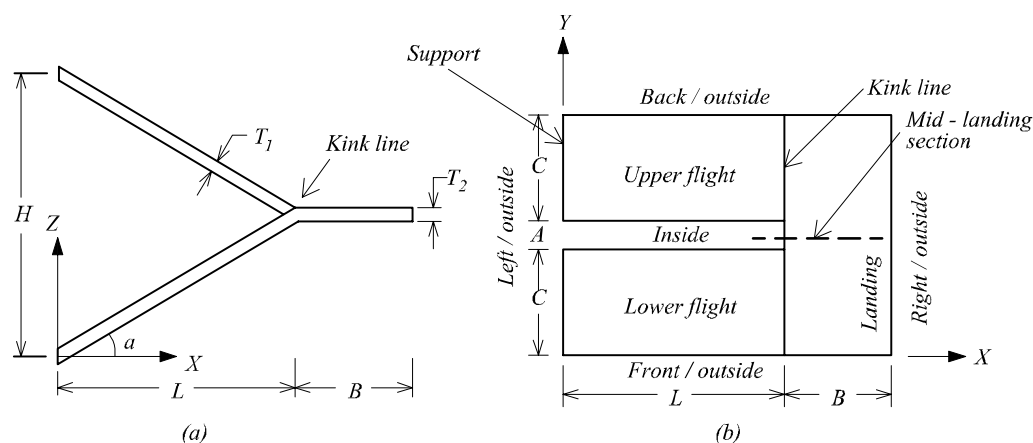


Figure 6.7.2.1: Free Standing Stair Slab Geometry, (a) Elevation, (b) Plan

6.7.2.2 Loading

Staircases shall be designed to support the design ultimate load according to the load combinations specified in Chapter 2, loads.

6.7.2.3 Distribution of Loading

The stair shall be designed for uniformly distributed loading.

6.7.2.4 Depth of Section

The depth of the section shall be taken as the minimum thickness perpendicular to the soffit of the staircase.

6.7.3 DESIGN

6.7.3.1.1 Empirical expressions for deflections, and forces and moments at critical locations of free standing stairs are given in terms of the various dimensions of the stairway in Table 6.7.3.1. These expressions, which are explicit and of empirical nature, are valid within the applicable range of the geometric parameters (see Figure 6.7.2.1) and concrete strength. In these equations, the unit of force is Kilo Newton (kN) and the unit of length is millimeter (mm). Thickness of the flight and landing slabs are assumed to be equal.

6.7.3.1.2 The equations in Table 6.7.3.1 give working values of moments and forces corresponding to 0.48×10^{-2} MPa live load and appropriate dead load of slab and steps based on unit weight of 2.356×10^{-5} N/mm³. Forces and moments for other values of live load shall be calculated by simple proportioning.

6.7.3.1.3 To convert from working to ultimate design values, the working values of moments and forces in Table 6.7.1 shall be multiplied by a conversion factor equal to the ratio of factored ultimate load and un-factored service load.

6.7.3.1.4 Apart from maintaining the standard code provisions in detailing the reinforcement as stipulated elsewhere in this code, additional detailing as described below (6.7.3.1.5 through 6.7.3.1.9) shall be done to take care of the important features which are special to the free standing stairway.

6.7.3.1.5 To account for the non-uniform distribution of the total bending moment at support across the width of the section, three-fourths of the total negative steel shall be distributed across the outer half of the width of support section and the rest of the negative steel shall be distributed within the inner half of the width of support section.

- 6.7.3.1.6 Similar proportioning of reinforcement layout as to 6.7.3.1.5, but in reverse order shall also be done at flight-landing junction (kink).
- 6.7.3.1.7 At midspan of flights, the positive steel shall be distributed uniformly across the section.
- 6.7.3.1.8 Of the total steel required to resist the negative bending at mid-landing section, 50 percent shall be placed within the inner one-third of the width of section. The rest shall be distributed across of the outer two-thirds of the width.
- 6.7.3.1.9 The suggested bar curtailment scheme for the free standing stairway is shown in Figure 6.7.3.1. Half of the negative steel at support may be terminated at a distance of $L/4$ from the support. Another 25 percent may be bent downward at a distance of $L/4$ to provide part of the flight midspan positive steel. The rest 25 percent is recommended to continue straight towards the flight-landing junction. This 25 percent may be merged with the negative steel at kink. Fifty percent of the flight midspan positive steel should span from kink and terminate at a point $L/5$ from the support unless they are bent up for negative steel. The rest should start from a point at a distance of $L/5$ from kink and would terminate at $L/4$ from support. Of the total negative steel at mid-landing section, half of it will terminate at a distance $C/2$ from free edge and the rest will cover the whole length of landing. Half of the negative steel at kink will project into landing upto the free edge and the rest may be terminated at a distance of $B/2$.

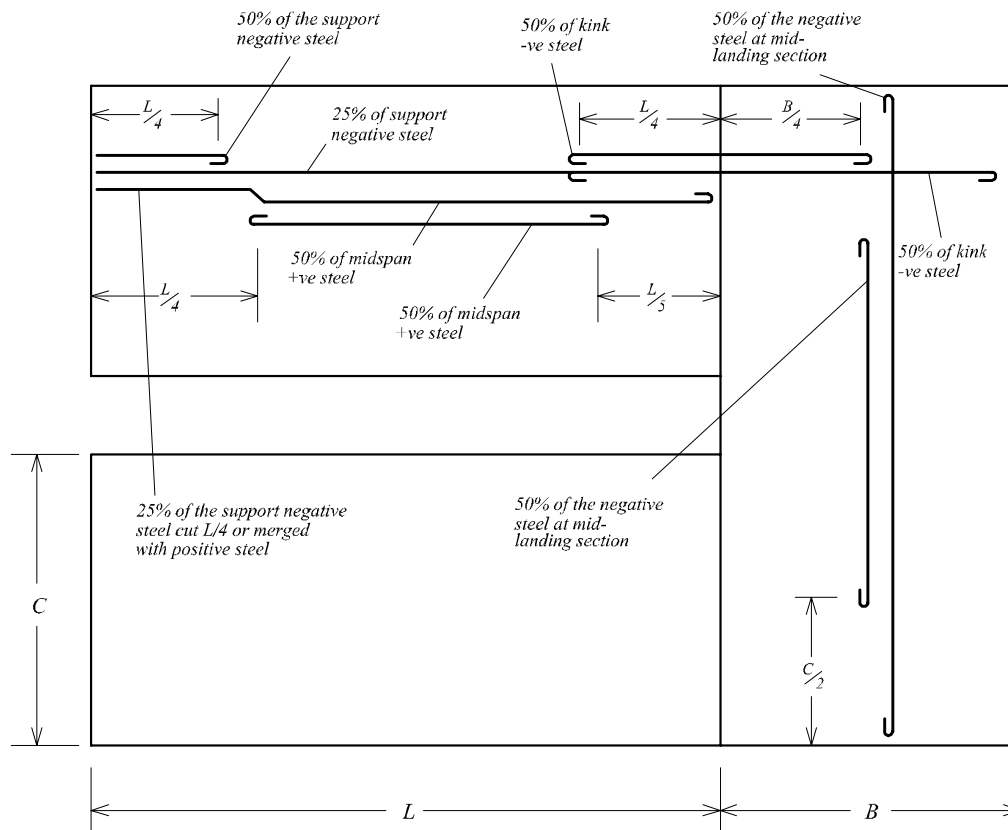


Figure 6.7.3.1: Recommended Bar Curtailment Details for Free Standing Stairs

Table 6.7.3.1 EXPRESSIONS FOR DEFLECTION, FORCES & MOMENTS IN FREE STANDING STAIRS

		$A = \text{_____ mm.}$ $150 < A < 1000$	$B = \text{_____ mm}$ $900 < B < 1875$	$C = \text{_____ mm.}$ $915 < C < 1900$	$L = \text{_____ mm.}$ $2030 < L < 3550$	$H = \text{_____ mm.}$ $2440 < H < 4320$	$T = \text{_____ mm.}$ $100 < T < 280$	f'_c = _____ MPa. $14 < f'_c < 40$	Value
Vertical deflection at landing corner, mm	Constant	$1+0.00545(A-125)^{0.94}$	$1+0.00114(B-914)^{1.1}$	$1+0.00165(C-914)^{0.93}$	$1-7.87 \times 10^{-6} (L-2030)$	$1-19.68 \times 10^{-6} (H-2440)$	$1-0.161 (T-100)^{0.334}$	$1 - 1.074 \times 10^{-6} (f'_c - 14)^{.93}$	
	2.03								
Support negative moment, kN-m	Constant	$1.555+0.000787(A-50)$	$1.06-0.00022 (B-860)$	$1.2+0.00276(C-864)$	$1+0.000748(L-2030)$	$1+5.9 \times 10^{-6} (H-2440)$	$0.39+0.00173(T-90)$	1.0	
	-4.712								
Flight midspan positive moment, kN-m	Constant	$1.1-31.48 \times 10^{-6} (A-150)^{1.52}$	$1-70.11 \times 10^{-6} (B-915)^{1.365}$	1.0	$1+0.128 \times 10^{-6} (L-2030)^{2.66}$	$1+0.899 \times 10^{-9} (H-2440)^{2.77}$	$1-0.00165(T-100)^{1.17}$	1	
	-1.526								
Negative moment at kink, kN-m	Constant	$1.23+0.000512(A-125)$	$1.01+0.00323(B-915)$	$.85+0.000709(C-915)$	1.0	1.0	$0.95+0.00447(T-100)^{1.03}$	1	
	-3.447								
Negative moment at midsection of landing, kN-m	Constant	$1+0.000303(A-150)$	$1+0.00118(B-915)$	$1+0.00106(C-915)$	$1+0.000409(L-2030)$	$1+26.37 \times 10^{-6} (H-2040)$	$1+0.00185(T-100)$	1	
	-6.14								
Axial force in flights, kN	Constant	$1+0.000236(A-125)$	$1+0.000787(B-915)$	$1+0.000827(C-915)$	$1+0.000354(L-2030)$	$1-0.000157(H-2440)$	$1+0.00276(T-100)$	1	
	34.69								

Torsion in flights, kN-m.	Constant	1+0.00177(A-125)	1+0.00063(B-915)	1+0.00268(C-915)	$1-8.0 \times 10^{-6}(L-2030)^{.75}$	1.0	1+0.00358(T-100)	1	
	2.312								
Inplane moment in flights, kN-m	Constant	1.1+0.000866(A-150)	1+0.000984(B-915)	1+0.00157(C-915)	1+0.00059(L-2030)	1-0.000197(H-2440)	1+0.0026(T-100)	1	
	-14.35								
Lateral shear in mid-section of landing, kN	Constant	1-0.000276(A-150)	1+0.00138(B-915)	1+0.000709(C-915)	1+0.000669(L-2030)	1-0.00024(H-2440)	1+0.000746(T-100) ^{1.3}	1	
	30.17								

6.7.4 Sawtooth (slab less) stair

6.7.4.1 Loading

The stair shall be designed to support the design ultimate load according to the load combinations specified in Chapter 2, Loads.

6.7.4.2 Distribution of loading

Where flights or landing are embedded at least 110 mm into the walls and are designed to span in the direction of the flight, a 150mm strip may be deducted from the loaded area and the effective breadth of the section may be increased by 75 mm for the purpose of design (Figure 6.7.1.3).

6.7.4.3 Effective span

Sawtooth stairs shall be supported with stringer beams at landing levels (Figure 6.7.4.1). The effective span for the stair shall be the going of the stair measured horizontally (Figure 6.7.4.1) from the face of the stringer beams.

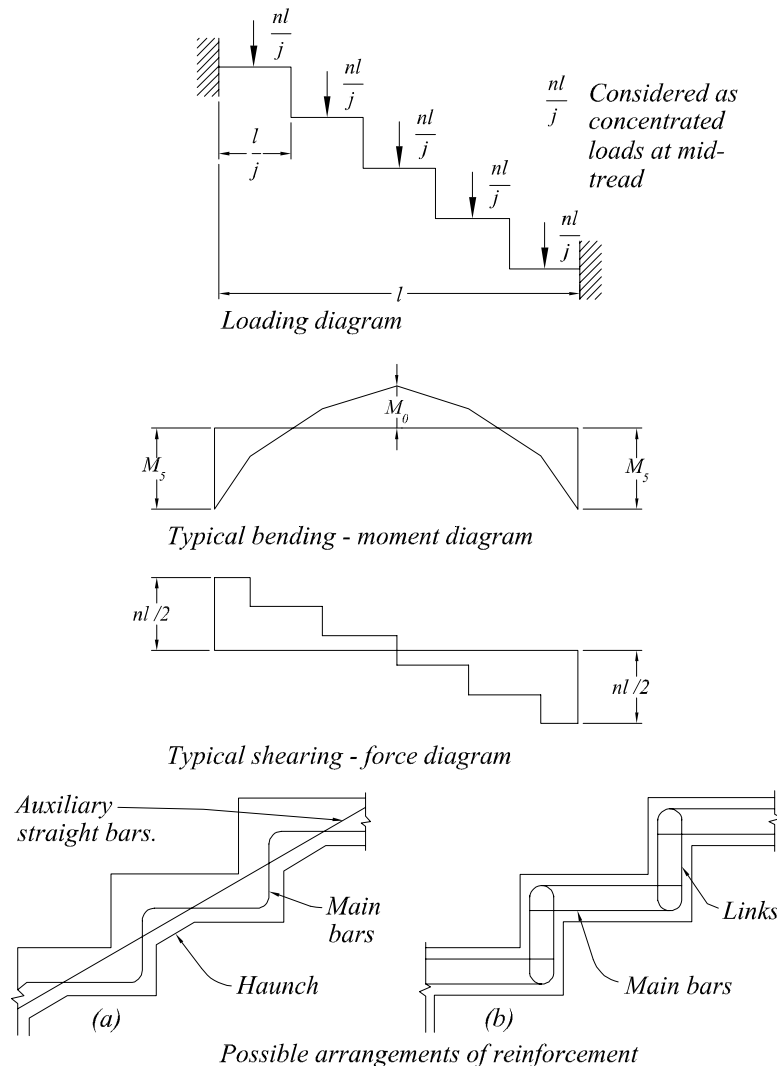


Figure 6.7.4.1: Elements of saw-tooth stair and typical reinforcement arrangements

6.7.4.4 Design

The midspan moments for sawtooth stairs are given by:

$$M_s = \frac{nl^2(k_{11} + k_0 k_{12})}{j^2(k_{13} + k_0 k_{14})} \quad (6.7.1)$$

Where, k_0 = stiffness of tread/stiffness of riser and j is the number of treads (Figure 6.7.4.1).

If j is odd: $k_{11} = \frac{1}{16}j^2 + \frac{1}{48}j(j-1)(j-2)$, $k_{12} = \frac{1}{16}(j-1)^2 + \frac{1}{48}j(j-1)(j-2)(j-3)$,
 $k_{13} = \frac{1}{2}j$, $k_{14} = \frac{1}{2}(j-1)$.

If j is even: $k_{11} = \frac{1}{48}j(j-1)(j-2)$, $k_{12} = \frac{1}{48}(j-1)(j-2)(j-3)$, $k_{13} = \frac{1}{2}(j-1)$,
 $k_{14} = \frac{1}{2}(j-2)$

The chart on Figure 6.7.4.2 gives the support-moment coefficients for various ratios of stiffness of tread/stiffness of riser and numbers of treads. Having found the support moment, the maximum midspan bending moment shall be determined by using the appropriate expression on the Figure 6.7.4.2 and subtracting the support moment.

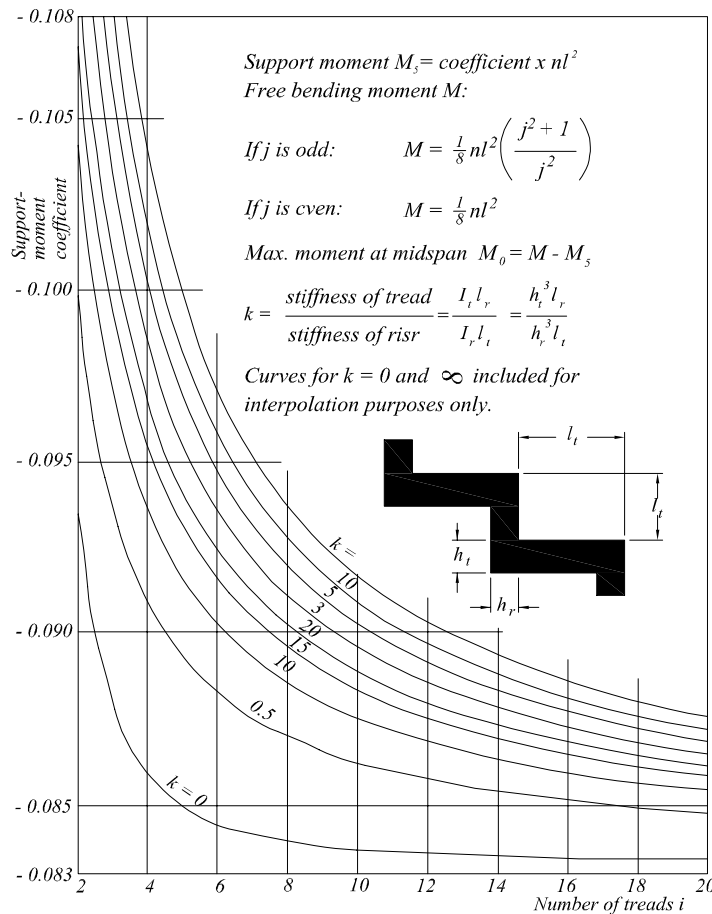


Figure 6.7.4.2: Support moment coefficients for saw-tooth stair

6.7.4.5 Detailing

Typical bending-moment and shearing-force diagrams for a stair are shown on Figure 6.7.4.1 together with suggested arrangements of reinforcement. The re-entrant corners of the stair-profile shall be designed for stress concentrations. This has to be facilitated by providing twice of the reinforcements calculated from Eqn. 6.7.1 and Figure 6.7.4.2. Fillets or haunches can also be incorporated in lieu at these junctions. The method of reinforcing the stair shown in diagram (a) of Figure 6.7.4.1 is very suitable but is generally only practicable if haunches are provided. Otherwise the arrangement shown in diagram (b) should be adopted.

6.7.5 Helicoidal stair

6.7.5.1 Loading

The stair shall be designed to support the design ultimate load according to the load combinations specified in Chapter 2, Loads.

6.7.5.2 Geometry

The pertinent geometry of the Helicoidal stair is given at Figure 6.7.5.1 where:

I_1, I_2 : second moment of area of stair section about horizontal axis and axis normal to slope, respectively

n : total loading per unit length projected along centre-line of load

R_1 : radius of centre-line of loading $= (2/3)(R_o^3 - R_i^3)/(R_o^2 - R_i^2)$

R_2 : radius of centre-line of steps $= (1/2)(R_i + R_o)$, where R_i and R_o are the internal and external radii of the stair, respectively

θ : angle subtended in plan between point considered and midpoint of stair

β : total angle subtended by helix in plan

ϕ : slope of tangent to helix centre-line measured from horizontal

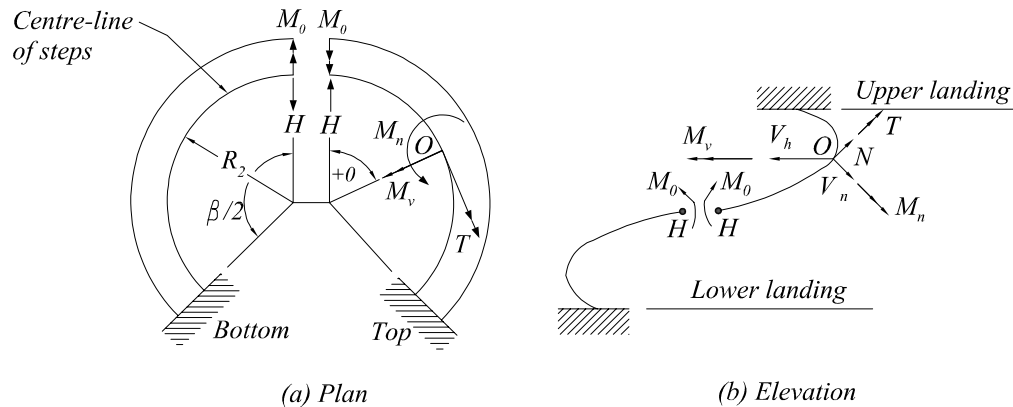


Figure 6.7.5.1: Elements of helicoidal stair (a) Plan; (b) Elevation

6.7.5.3 Effective span

Helicoidal stairs shall be supported with stringer beams at landing levels (Figure 6.7.1.1). The effective span for the stair shall be the total angle, β subtended by helix in plan measured horizontally (Figure 6.7.5.1) from the face of the stringer beams.

6.7.5.4 Depth of section

The depth of the section shall be taken as the minimum thickness perpendicular to the soffit of the stair unless otherwise the large geometric dimensions warrant calculating the deflections through a suitable numerical analysis.

6.7.5.5 Design

The design forces and moments for helicoidal stairs are given in Figure 6.7.5.2. Typical shear force, thrust, moment and torsion diagrams are provided in Figure 6.7.5.2. The moments, thrust, torsion and shear forces shall be obtained from the following equations:

Lateral moment:

$$M_n = M_o \sin \theta \sin \phi - HR_2 \theta \tan \phi \cos \theta \sin \phi - HR_2 \sin \theta \cos \phi + nR_1 \sin \phi (R_1 \sin \theta - R_2 \theta) \quad (6.7.2)$$

Torsional moment

$$T = (M_o \sin \theta - HR_2 \theta \cos \theta \tan \phi + nR_1^2 \sin \theta - nR_1 R_2 \theta) \cos \phi + HR_2 \sin \theta \sin \phi \quad (6.7.3)$$

Vertical moment:

$$M_y = M_o \cos \theta + HR_2 \theta \tan \phi \sin \theta - nR_1^2 (1 - \cos \theta) \quad (6.7.4)$$

Thrust:

$$N = -H \sin \theta \cos \phi + nR_1 \theta \sin \phi \quad (6.7.5)$$

Lateral shearing force across stair:

$$V_n = nR_1 \theta \cos \phi - H \sin \theta \sin \phi \quad (6.7.6)$$

Radial horizontal shearing force:

$$V_h = H \cos \theta \quad (6.7.7)$$

Where,

$$\text{Redundant moment acting tangentially at midspan: } M_o = k_1 n R_2^2 \quad (6.7.8)$$

$$\text{Horizontal redundant force at midspan: } H = k_2 n R_2 \quad (6.7.9)$$

$$\text{Vertical moment at supports: } M_{vs} = k_3 n R_2^2 \quad (6.7.10)$$

Values of coefficients k_1, k_2, k_3 for different $b/h, R_1/R_2$ and β , shall be obtained from Figures 6.7.5.3a-d.

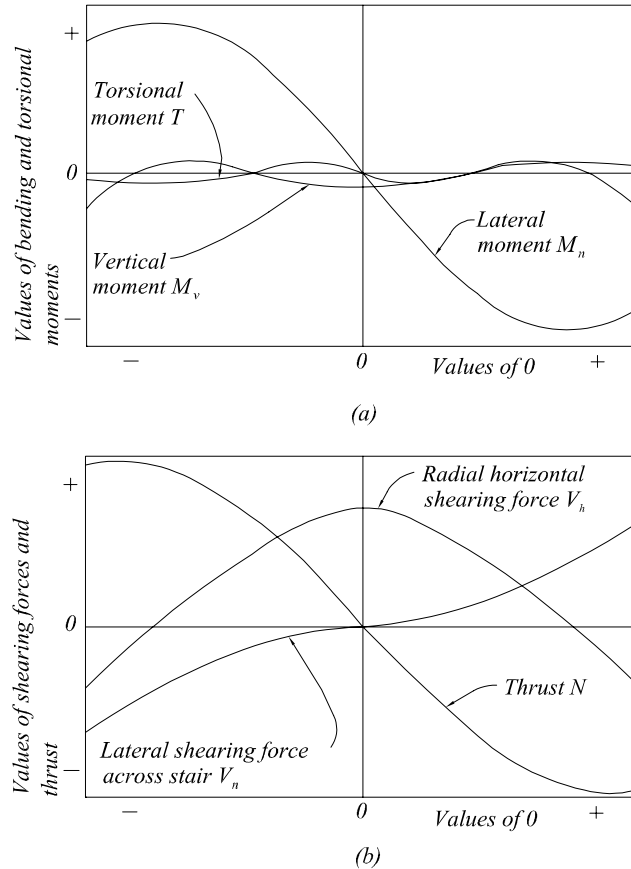


Figure 6.7.5.2: Typical force, moment and torsion diagrams for helicoidal stair.

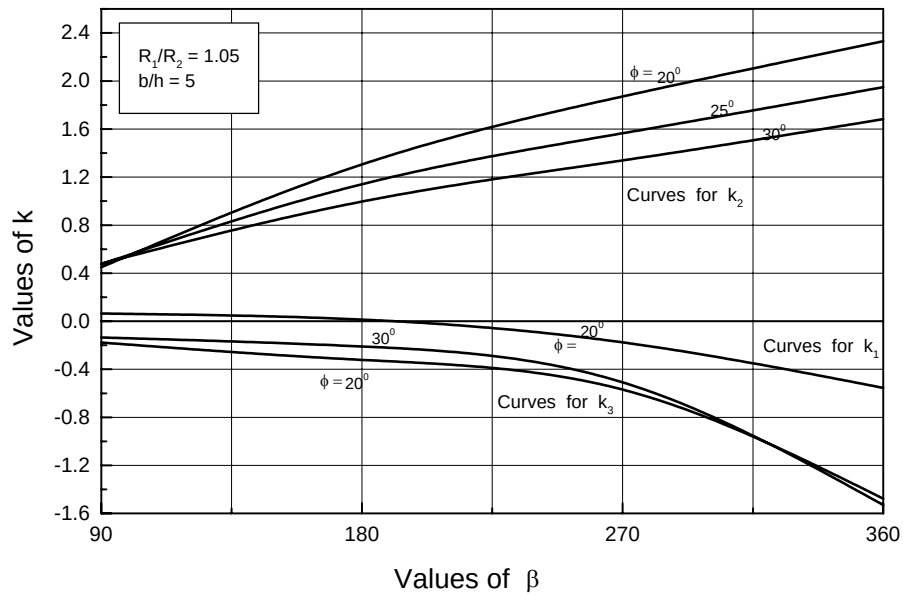


Figure 6.7.5.3a: Design charts for helicoidal stair slabs ($R_1/R_2 = 1.05$; $b/h = 5$)

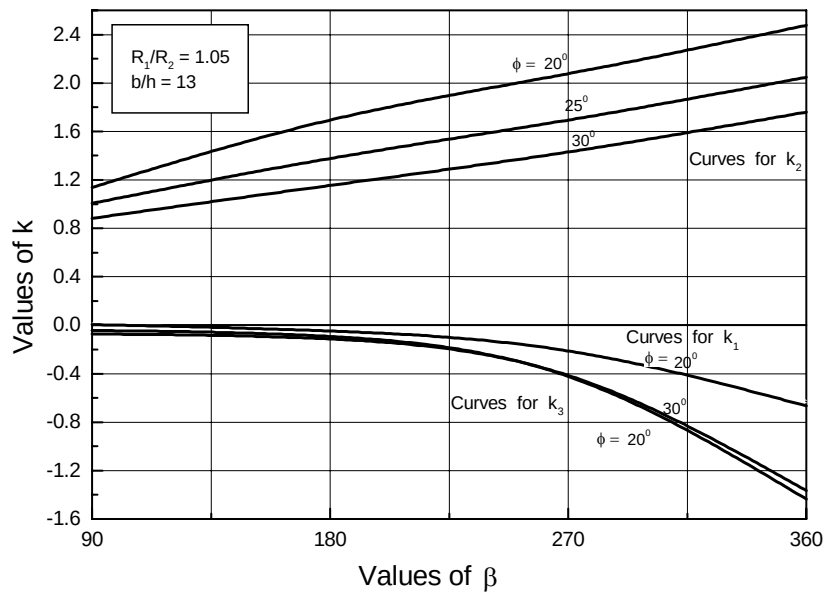


Figure 6.7.5.3b: Design charts for helicoidal stair slabs ($R_1/R_2 = 1.05$; $b/h = 13$)

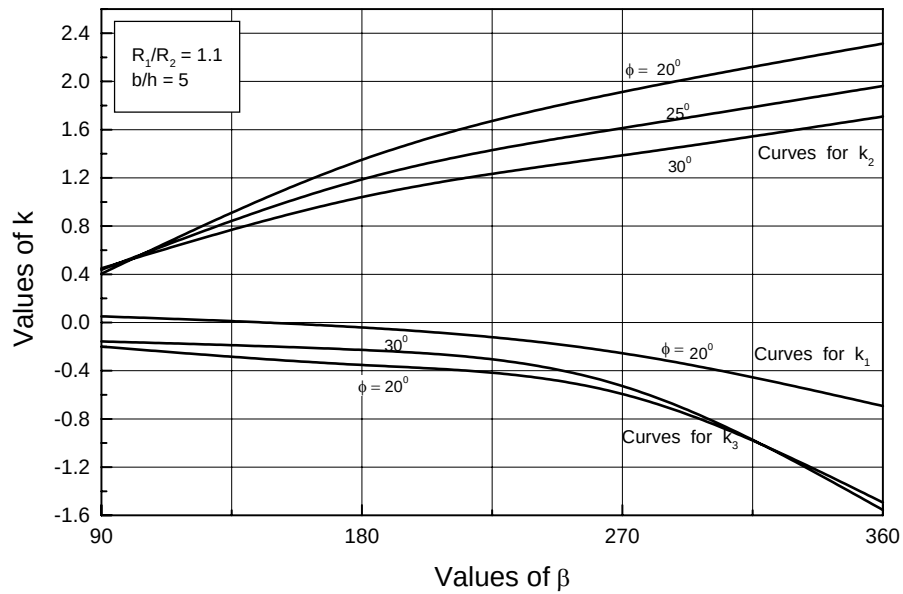


Figure 6.7.5.3c: Design charts for helicoidal stair slabs ($R_1/R_2 = 1.1$; $b/h = 5$)

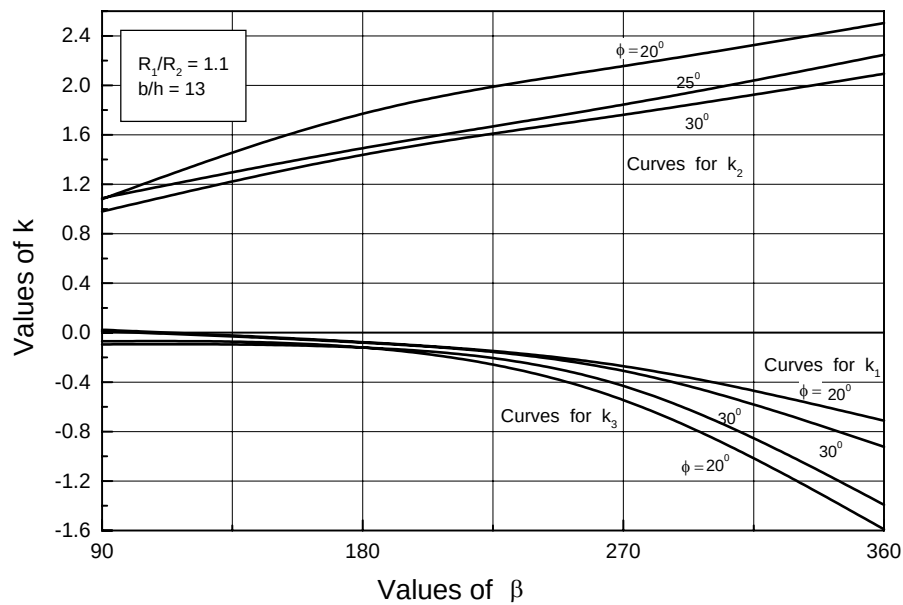


Figure 6.7.5.3d: Design charts for helicoidal stair slabs ($R_1/R_2 = 1.1$; $b/h = 13$)

6.8 FOOTINGS

6.8.1 Scope

6.8.1.1 Provisions of Sec. 6.8 shall apply for design of isolated footings and, where applicable, to combined footings and mats.

6.8.1.2 Additional requirements for design of combined footings and mats are given in 6.8.10.

6.8.2 Loads and reactions

6.8.2.1 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 Sec. 6.8.

6.8.2.2 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 determined through principles of soil mechanics.

6.8.2.3 For footings on piles, computations for moments and shears shall be permitted to be based on the assumption that the reaction from any pile is concentrated at pile center.

6.8.3 Equivalent square shapes for circular or regular polygon-shaped columns or pedestals supported by footings

For location of critical sections for moment, shear, and development of reinforcement in footings, it shall be permitted to treat circular or regular polygon-shaped concrete columns or pedestals as square members with the same area.

6.8.4 Moment in footings

6.8.4.1 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.

6.8.4.2 Maximum factored moment, M_u , for an isolated footing shall be computed as prescribed in 6.8.4.1 at critical sections located as follows:

- At face of column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall;
- Halfway between middle and edge of wall, for footings supporting a masonry wall;
- Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.

6.8.4.3 In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

6.8.4.4 In two-way rectangular footings, reinforcement shall be distributed in accordance with 6.8.4.4.1 and 6.8.4.4.2.

6.8.4.4.1 Reinforcement in long direction shall be distributed uniformly across entire width of footing.

6.8.4.4.2 For reinforcement in short direction, a portion of the total reinforcement, $\gamma_s A_s$, 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 $(1 - \gamma_s) A_s$, shall be distributed uniformly outside center band width of footing.

$$\gamma_s = \frac{2}{(\beta + 1)}$$

(6.8.1)

where β is ratio of long to short sides of footing.

6.8.5 Shear in footings

- 6.8.5.1 Shear strength of footings supported on soil or rock shall be in accordance with 6.4.10.
- 6.8.5.2 Location of critical section for shear in accordance with Sec. 6.4 shall be measured from face of column, pedestal, or wall, for footings supporting a 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 6.8.4.2(c).
- 6.8.5.3 Where the distance between the axis of any pile to the axis of the column is more than two times the distance between the top of the pile cap and the top of the pile, the pile cap shall satisfy 6.4.10 and 6.8.5.4. Other pile caps shall satisfy either Appendix A, or both 6.4.10 and 6.8.5.4. If Appendix A is used, the effective concrete compression strength of the struts, f_{ce} , shall be determined using A.3.2.2(b).
- 6.8.5.4 Computation of shear on any section through a footing supported on piles (Fig. 6.8.5.1) shall be in accordance with 6.8.5.4.1, 6.8.5.4.2, and 6.8.5.4.3.
- 6.8.5.4.1 Entire reaction from any pile with its center located $d_{pile}/2$ or more outside the section shall be considered as producing shear on that section.
- 6.8.5.4.2 Reaction from any pile with its center located $d_{pile}/2$ or more inside the section shall be considered as producing no shear on that section.
- 6.8.5.4.3 For intermediate positions of pile center, 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 $d_{pile}/2$ outside the section and zero value at $d_{pile}/2$ inside the section.

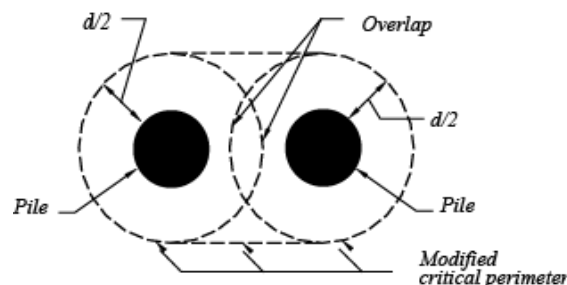


Fig. 6.8.5.1—Modified critical perimeter for shear with over- lapping critical perimeters.

6.8.6 Development of reinforcement in footings

- 6.8.6.1 Development of reinforcement in footings shall be in accordance with Sec. 8.2.
- 6.8.6.2 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.

- 6.8.6.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in 6.8.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. See also 8.2.7.6.

6.8.7 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.

6.8.8 Force transfer at base of column, wall, or reinforced pedestal

- 6.8.8.1 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.

- 6.8.8.1.1 Bearing stress on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by 6.3.14.

- 6.8.8.1.2 Reinforcement, dowels, or mechanical connectors between supported and supporting 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 6.8.8.2 or 6.8.8.3.

- 6.8.8.1.3 If calculated moments are transferred to supporting pedestal or footing, then reinforcement, dowels, or mechanical connectors shall be adequate to satisfy 8.2.15.

- 6.8.8.1.4 Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of 6.4.5, or by other appropriate means.

- 6.8.8.2 In cast-in-place construction, reinforcement required to satisfy 6.8.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

- 6.8.8.2.1 For cast-in-place columns and pedestals, area of reinforcement across interface shall be not less than $0.005A_g$, where A_g is the gross area of the supported member.

- 6.8.8.2.2 For cast-in-place walls, area of reinforcement across interface shall be not less than minimum vertical reinforcement given in 6.6.3.2.

- 6.8.8.2.3 At footings, it shall be permitted to lap splice $\Phi 43$ mm and $\Phi 57$ mm longitudinal bars, in compression only, with dowels to provide reinforcement required to satisfy 6.8.8.1. Dowels shall not be larger than $\Phi 36$ mm bar and shall extend into supported member a distance not less than the larger of l_{dc} , of $\Phi 43$ mm or $\Phi 57$ mm bars and compression lap splice length of the dowels, whichever is greater, and into the footing a distance not less than l_{dc} of the dowels.

- 6.8.8.2.4 If a pinned or rocker connection is provided in cast-in-place construction, connection shall conform to 6.8.8.1 and 6.8.8.3.

- 6.8.8.3 In precast construction, anchor bolts or suitable mechanical connectors shall be permitted for satisfying 6.8.8.1. Anchor bolts shall be designed in accordance with Appendix D.

- 6.8.8.3.1 Connection between precast columns or pedestals and supporting members shall meet the requirements of 6.10.5.1.3(a).

- 6.8.8.3.2 Connection between precast walls and supporting members shall meet the requirements of 6.10.5.1.3(b) and (c).

6.8.8.3.3 Anchor bolts and mechanical connections shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete. Anchor bolts shall be designed in accordance with Appendix D.

6.8.9 Stepped or sloped footings

6.8.9.1 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 8.2.7.6.)

6.8.9.2 Sloped or stepped footings designed as a unit shall be constructed to ensure action as a unit.

6.8.10 Combined footings and mats

6.8.10.1 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 the code.

6.8.10.2 The direct design method of Sec. 6.5 shall not be used for design of combined footings and mats.

6.8.10.3 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.

6.8.10.4 Minimum reinforcing steel in mat foundations shall meet the requirements of Sec. 8.1.11.2 in each principal direction. Maximum spacing shall not exceed 450 mm.

6.9 FOLDED PLATES AND SHELLS

6.9.1 Scope and definitions

6.9.1.1 Provisions of Sec. 6.9 shall apply to thin shell and folded plate concrete structures, including ribs and edge members.

6.9.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Sec. 6.9, shall apply to thin-shell structures.

6.9.1.3 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 behavior, 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.

6.9.1.4 Folded plates

A class of shell structure formed by joining flat, thin slabs along their edges to create a three-dimensional spatial structure.

6.9.1.5 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.

6.9.1.6 Auxiliary members

Ribs or edge beams that serve to strengthen, stiffen, or support the shell; usually, auxiliary members act jointly with the shell.

6.9.1.7 Elastic analysis

An analysis of deformations and internal forces based on equilibrium, compatibility of strains, and assumed elastic behavior, and representing to a suitable approximation the three-dimensional action of the shell together with its auxiliary members.

6.9.1.8 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 a suitable approximation three-dimensional action of the shell together with its auxiliary members.

6.9.1.9 Experimental analysis

An analysis procedure based on the measurement of deformations or strains, or both, of the structure or its model; experimental analysis is based on either elastic or inelastic behavior.

6.9.2 Analysis and design

- 6.9.2.1 Elastic behavior shall be an accepted basis for determining internal forces and displacements of thin shells. This behavior shall be permitted to be established by computations based on an analysis of the uncracked concrete structure in which the material is assumed linearly elastic, homogeneous, and isotropic. Poisson's ratio of concrete shall be permitted to be taken equal to zero.
- 6.9.2.2 Inelastic analyses shall be permitted to be used where it can be shown that such methods provide a safe basis for design.
- 6.9.2.3 Equilibrium checks of internal resistances and external loads shall be made to ensure consistency of results.
- 6.9.2.4 Experimental or numerical analysis procedures shall be permitted where it can be shown that such procedures provide a safe basis for design.
- 6.9.2.5 Approximate methods of analysis shall be permitted where it can be shown that such methods provide a safe basis for design.
- 6.9.2.6 The thickness of a shell and its reinforcement shall be proportioned for the required strength and serviceability, using either the strength design method of 6.1.2.1 or the design method of 6.1.2.2.
- 6.9.2.7 Shell instability shall be investigated and shown by design to be precluded.
- 6.9.2.8 Auxiliary members shall be designed according to the applicable provisions of the Code. It shall be permitted to assume that a portion of the shell equal to the flange width, as specified in 6.1.13, acts 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 6.1.13.5.
- 6.9.2.9 Strength design of shell slabs for membrane and bending forces shall be based on the distribution of stresses and strains as determined from either an elastic or an inelastic analysis.
- 6.9.2.10 In a region where membrane cracking is predicted, the nominal compressive strength parallel to the cracks shall be taken as $0.4f'_c$.

6.9.3 Design strength of materials

- 6.9.3.1 Specified compressive strength of concrete f'_c at 28 days shall not be less than 21 MPa.
- 6.9.3.2 Specified yield strength of reinforcement f_y shall not exceed 420 MPa.

6.9.4 Shell reinforcement

- 6.9.4.1 Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces, to resist tension from bending and twisting moments, to limit shrinkage and temperature crack width and spacing, and as reinforcement at shell boundaries, load attachments, and shell openings.
- 6.9.4.2 Tensile reinforcement shall be provided in two or more directions and shall be proportioned such that its resistance in any direction equals or exceeds the component of internal forces in that direction. Alternatively, reinforcement for the membrane forces in the slab shall be calculated as the reinforcement required to resist axial tensile forces plus the tensile force due to shear-friction required to transfer shear across any cross section of the membrane. The assumed coefficient of friction, μ , shall not exceed that specified in 6.4.5.4.3.
- 6.9.4.3 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 8.1.11.
- 6.9.4.4 Reinforcement for shear and bending moments about axes in the plane of the shell slab shall be calculated in accordance with Sec. 6.3, 6.4 and 6.5.
- 6.9.4.5 The area of shell tension reinforcement shall be limited so that the reinforcement will yield before either crushing of concrete in compression or shell buckling can take place.
- 6.9.4.6 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, it shall be permitted to place membrane reinforcement in two or more component directions.
- 6.9.4.7 If the direction of reinforcement varies more than 10 degrees from the direction of principal tensile membrane force, the amount of reinforcement shall be reviewed in relation to cracking at service loads.
- 6.9.4.8 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 shall be permitted to 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.
- 6.9.4.9 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.
- 6.9.4.10 Shell reinforcement in any direction shall not be spaced farther apart than 450 mm nor farther apart than five times the shell thickness. Where the principal membrane tensile stress on the gross concrete area due to factored loads exceeds $0.33\phi\lambda\sqrt{f'_c}$, reinforcement shall not be spaced farther apart than three times the shell thickness.
- 6.9.4.11 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 Sec. 8.2, except that the minimum development length shall be $1.2l_d$ but not less than 450 mm.

- 6.9.4.12 Splice lengths of shell reinforcement shall be governed by the provisions of Sec. 8.2, except that the minimum splice length of tension bars shall be 1.2 times the value required by Sec. 8.2 but not less than 450 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.

6.9.5 Construction

- 6.9.5.1 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 , used 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.
- 6.9.5.2 Contract documents 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 behavior.

6.10 PRECAST CONCRETE

6.10.1 Scope

- 6.10.1.1 All provisions of this Code, not specifically excluded and not in conflict with the provisions of Sec. 6.10, shall apply to structures incorporating precast concrete structural members.

6.10.2 General

- 6.10.2.1 Design of precast members and connections shall include loading and restraint conditions from initial fabrication to end use in the structure, including form removal, storage, transportation, and erection.
- 6.10.2.2 When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.
- 6.10.2.3 Tolerances for both precast members and interfacing members shall be specified. Design of precast members and connections shall include the effects of these tolerances.
- 6.10.2.4 In addition to the requirements for drawings and specifications in Sec. 1.9, (a) and (b) shall be included in either the contract documents or shop drawings:
- Details of reinforcement, inserts and lifting devices required to resist temporary loads from handling, storage, transportation, and erection;
 - Required concrete strength at stated ages or stages of construction.

6.10.3 Distribution of forces among members

- 6.10.3.1 Distribution of forces that are perpendicular to the plane of members shall be established by analysis or by test.
- 6.10.3.2 Where the system behavior requires in-plane forces to be transferred between the members of a precast floor or wall system, 6.10.3.2.1 and 6.10.3.2.2 shall apply.
- 6.10.3.2.1 In-plane force paths shall be continuous through both connections and members.
- 6.10.3.2.2 Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

6.10.4 Member design

- 6.10.4.1 In one-way precast floor and roof slabs and in one-way precast, prestressed wall panels, all not wider than 3.7 m, and where members are not mechanically connected to cause restraint in the transverse direction, the shrinkage and temperature reinforcement requirements of Sec. 8.1.11 in the direction normal to the flexural reinforcement shall be permitted to be waived. This waiver shall not apply to members that require reinforcement to resist transverse flexural stresses.
- 6.10.4.2 For precast, nonprestressed walls the reinforcement shall be designed in accordance with the provisions of Sec. 6.3 or 6.6, except that the area of horizontal and vertical reinforcement each shall be not less than $0.001A_g$, where A_g is the gross cross-sectional area of the wall panel. Spacing of reinforcement shall not exceed 5 times the wall thickness nor 750 mm for interior walls nor 450 mm for exterior walls.
- 6.10.5 Structural integrity**
- 6.10.5.1 Except where the provisions of 6.10.5.2 govern, the minimum provisions of 6.10.5.1.1 through 6.10.5.1.4 for structural integrity shall apply to all precast concrete structures.
- 6.10.5.1.1 Longitudinal and transverse ties required by Sec. 8.1.12.3 shall connect members to a lateral load-resisting system.
- 6.10.5.1.2 Where precast elements form floor or roof diaphragms, the connections between diaphragm and those members being laterally supported shall have a nominal tensile strength capable of resisting not less than 4.4 kN per linear m.
- 6.10.5.1.3 Vertical tension tie requirements of Sec. 8.1.12.3 shall apply to all vertical structural members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with (a) through (c):
- a) Precast columns shall have a nominal strength in tension not less than $1.4A_g$, in lb. For columns with a larger cross section than required by consideration of loading, a reduced effective area A_g , based on cross section required but not less than one-half the total area, shall be permitted;
 - b) Precast wall panels shall have a minimum of two ties per panel, with a nominal tensile strength not less than 44 kN per tie;
 - c) When design forces result in no tension at the base, the ties required by 6.10.5.1.3(b) shall be permitted to be anchored into an appropriately reinforced concrete floor slab-on-ground.
- 6.10.5.1.4 Connection details that rely solely on friction caused by gravity loads shall not be used.
- 6.10.5.2 For precast concrete bearing wall structures three or more stories in height, the minimum provisions of 6.10.5.2.1 through 6.10.5.2.5 shall apply (Fig. 6.10.5.1).
- 6.10.5.2.1 Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 22 kN per meter of width or length. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 600 mm of the plane of the floor or roof system.

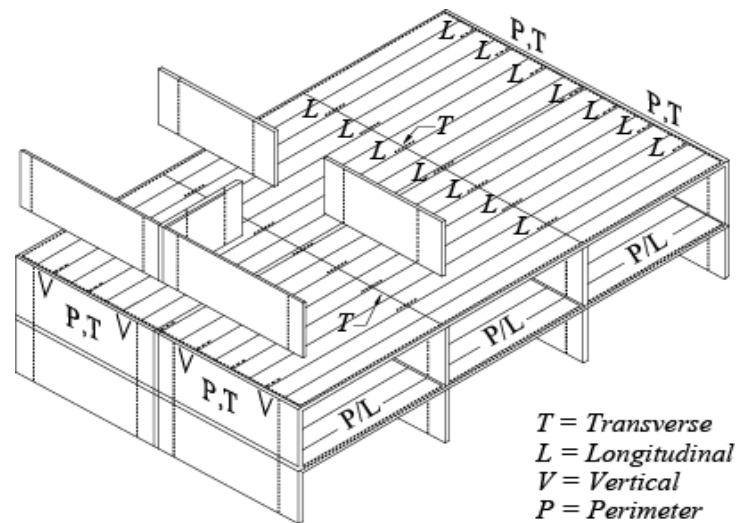


Fig. 6.10.5.1—Typical arrangement of tensile ties in large panel structures.

- 6.10.5.2.2 Longitudinal ties parallel to floor or roof slab spans shall be spaced not more than 3 m on centers. Provisions shall be made to transfer forces around openings.
- 6.10.5.2.3 Transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.
- 6.10.5.2.4 Ties around the perimeter of each floor and roof, within 1.2 m of the edge, shall provide a nominal strength in tension not less than 71 kN.
- 6.10.5.2.5 Vertical tension ties shall be provided in all walls and shall be continuous over the height of the building. They shall provide a nominal tensile strength not less than 44 kN per horizontal meter of wall. Not less than two ties shall be provided for each precast panel.

6.10.6 Connection and bearing design

- 6.10.6.1 Forces shall be permitted to be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing steel connections, reinforced topping, or a combination of these means.
 - 6.10.6.1.1 The adequacy of connections to transfer forces between members shall be determined by analysis or by test. Where shear is the primary result of imposed loading, it shall be permitted to use the provisions of 6.4.5 as applicable.
 - 6.10.6.1.2 When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.
- 6.10.6.2 Bearing for precast floor and roof members on simple supports shall satisfy 6.10.6.2.1 and 6.10.6.2.2.
 - 6.10.6.2.1 The allowable bearing stress at the contact surface between supported and supporting members and between any intermediate bearing elements shall not exceed the bearing strength for either surface or the bearing element, or both. Concrete bearing strength shall be as given in 6.3.14.

6.10.6.2.2 Unless shown by test or analysis that performance will not be impaired, (a) and (b) shall be met (Fig. 6.10.6.1):

- a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least $l_n/180$, but not less than:

For solid or hollow-core slabs 50 mm

For beams or stemmed members 75 mm

- b) Bearing pads at unarmored edges shall be set back a minimum of 13 mm from the face of the support, or at least the chamfer dimension at chamfered edges.

6.10.6.2.3 The requirements of 8.2.8.1 shall not apply to the positive bending moment reinforcement for statically determinate precast members, but at least one-third of such reinforcement shall extend to the center of the bearing length, taking into account permitted tolerances in Sec. 8.1.5.2c and 6.10.2.3.

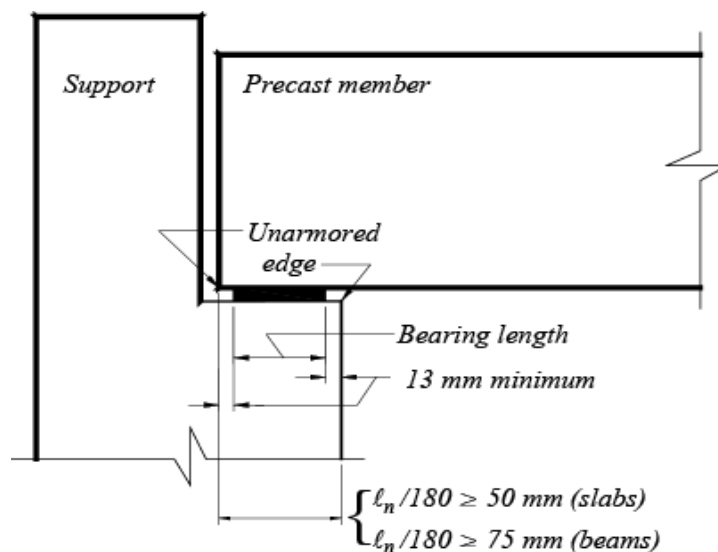


Fig. 6.10.6.1—Bearing length on support

6.10.7 Items embedded after concrete placement

6.10.7.1 When approved by the designer, embedded items (such as dowels or inserts) that either protrude from the concrete or remain exposed for inspection shall be permitted to be embedded while the concrete is in a plastic state provided that 6.10.7.1.1, 6.10.7.1.2, and 6.10.7.1.3 are met.

6.10.7.1.1 Embedded items are not required to be hooked or tied to reinforcement within the concrete.

6.10.7.1.2 Embedded items are maintained in the correct position while the concrete remains plastic.

6.10.7.1.3 The concrete is properly consolidated around the embedded item.

6.10.8 Marking and identification

6.10.8.1 Each precast member shall be marked to indicate its location and orientation in the structure and date of manufacture.

6.10.8.2 Identification marks shall correspond to placing drawings.

6.10.9 Handling

6.10.9.1 Member design shall consider forces and distortions during curing, stripping, storage, transportation, and erection so that precast members are not overstressed or otherwise damaged.

6.10.9.2 During erection, precast members and structures shall be adequately supported and braced to ensure proper alignment and structural integrity until permanent connections are completed.

6.10.10 Evaluation of strength of precast construction

6.10.10.1 A precast element to be made composite with cast-in-place concrete shall be permitted to be tested in flexure as a precast element alone in accordance with 6.10.10.1.1 and 6.10.10.1.2.

6.10.10.1.1 Test loads shall be applied only when calculations indicate the isolated precast element will not be critical in compression or buckling.

6.10.10.1.2 The test load shall be that load which, when applied to the precast member alone, induces the same total force in the tension reinforcement as would be induced by loading the composite member with the test load required by 6.11.3.2.

6.10.10.2 The provisions of 6.11.5 shall be the basis for acceptance or rejection of the precast element.

6.11 EVALUATION OF STRENGTH OF EXISTING STRUCTURES

6.11.1 Strength evaluation — General

6.11.1.1 If there is doubt that a part or all of a structure meets the safety requirements of this Code, a strength evaluation shall be carried out as required by the Engineer.

6.11.1.2 If the effect of the strength deficiency is well understood and if it is feasible to measure the dimensions and material properties required for analysis, analytical evaluations of strength based on those measurements shall suffice. Required data shall be determined in accordance with 6.11.2.

6.11.1.3 If the effect of the strength deficiency is not well understood or if it is not feasible to establish the required dimensions and material properties by measurement, a load test shall be required if the structure is to remain in service.

6.11.1.4 If the doubt about safety of a part or all of a structure involves deterioration, and if the observed response during the load test satisfies the acceptance criteria, the structure or part of the structure shall be permitted to remain in service for a specified time period. If deemed necessary by the Engineer, periodic reevaluations shall be conducted.

6.11.2 Determination of material properties and required dimensions

6.11.2.1 Dimensions of the structural elements shall be established at critical sections.

6.11.2.2 Locations and sizes of the reinforcing bars, welded wire reinforcement, or tendons shall be determined by measurement. It shall be permitted to base reinforcement locations on available drawings if spot checks are made confirming the information on the drawings.

6.11.2.3 If required, concrete strength shall be based on results of cylinder tests from the original construction or tests of cores removed from the part of the structure where the strength is in question. For strength evaluation of an existing structure, cylinder or core test data shall be used to estimate an equivalent f'_c . The method for obtaining and testing cores shall be in accordance with ASTM C42M.

6.11.2.4 If required, reinforcement or prestressing steel strength shall be based on tensile tests of representative samples of the material in the structure in question.

6.11.2.5 If the required dimensions and material properties are determined through measurements and testing, and if calculations can be made in accordance with 6.11.1.2, it shall be permitted to increase ϕ from those specified in 6.2.3, but ϕ shall not be more than:

Tension-controlled sections, as defined in 6.3.3.4	1.0
Compression-controlled sections, as defined in 6.3.3.3:	
Members with spiral reinforcement conforming to 6.3.9.3	0.9
Other reinforced members	0.8
Shear and/or torsion	0.8
Bearing on concrete	0.8

6.11.3 Load test procedure

6.11.3.1 Load arrangement

The number and arrangement of spans or panels loaded shall be selected to maximize the deflection and stresses in the critical regions of the structural elements of which strength is in doubt. More than one test load arrangement shall be used if a single arrangement will not simultaneously result in maximum values of the effects (such as deflection, rotation, or stress) necessary to demonstrate the adequacy of the structure.

6.11.3.2 Load intensity

The total test load (including dead load already in place) shall not be less than the larger of (a), (b), and (c):

- a) $1.15D + 1.5L + 0.4(L_r \text{ or } P)$
- b) $1.15D + 0.9L + 1.5(L_r \text{ or } P)$
- c) $1.3D$

The load factor on the live load L in (b) shall be permitted to be reduced to 0.45 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 4.8 kN/m^2 . It shall be permitted to reduce L in accordance with the provisions of the applicable general building code.

6.11.3.3 A load test shall not be made until that portion of the structure to be subjected to load is at least 56 days old. If the owner of the structure, the contractor, and all involved parties agree, it shall be permitted to make the test at an earlier age.

6.11.4 Loading criteria

6.11.4.1 The initial value for all applicable response measurements (such as deflection, rotation, strain, slip, crack widths) shall be obtained not more than 1 hour before application of the first load increment. Measurements shall be made at locations where maximum response is expected. Additional measurements shall be made if required.

6.11.4.2 Test load shall be applied in not less than four approximately equal increments.

6.11.4.3 Uniform test load shall be applied in a manner to ensure uniform distribution of the load transmitted to the structure or portion of the structure being tested. Arching of the applied load shall be avoided.

6.11.4.4 A set of response measurements shall be made after each load increment is applied and after the total load has been applied on the structure for at least 24 hours.

6.11.4.5 Total test load shall be removed immediately after all response measurements defined in 6.11.4.4 are made.

6.11.4.6 A set of final response measurements shall be made 24 hours after the test load is removed.

6.11.5 Acceptance criteria

6.11.5.1 The portion of the structure tested shall show no evidence of failure. Spalling and crushing of compressed concrete shall be considered an indication of failure.

6.11.5.2 Measured deflections shall satisfy either Eq. (6.11.1) or (6.11.2):

$$\Delta_1 \leq \frac{\ell_t^2}{20,000h} \quad (6.11.1)$$

$$\Delta_r \leq \frac{\Delta_1}{4} \quad (6.11.2)$$

If the measured maximum and residual deflections, Δ_1 and Δ_r , do not satisfy Eq. (6.11.1) or (6.11.2), it shall be permitted to repeat the load test.

The repeat test shall be conducted not earlier than 72 hours after removal of the first test load. The portion of the structure tested in the repeat test shall be considered acceptable if deflection recovery Δ_r satisfies the condition:

$$\Delta_r \leq \frac{\Delta_2}{5} \quad (6.11.3)$$

where Δ_2 is the maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test.

6.11.5.3 Structural members tested shall not have cracks indicating the imminence of shear failure.

6.11.5.4 In regions of structural members without transverse reinforcement, appearance of structural cracks inclined to the longitudinal axis and having a horizontal projection longer than the depth of the member at midpoint of the crack shall be evaluated.

6.11.5.5 In regions of anchorage and lap splices, the appearance along the line of reinforcement of a series of short inclined cracks or horizontal cracks shall be evaluated.

6.11.6 Provision for lower load rating

If the structure under investigation does not satisfy conditions or criteria of 6.11.1.2, 6.11.5.2, or 6.11.5.3, the structure shall be permitted for use at a lower load rating based on the results of the load test or analysis, if approved by the Engineer.

6.11.7 Safety

6.11.7.1 Load tests shall be conducted in such a manner as to provide for safety of life and structure during the test.

6.11.7.2 Safety measures shall not interfere with load test procedures or affect results.

6.12 COMPOSITE CONCRETE FLEXURAL MEMBERS**6.12.1 Scope**

6.12.1.1 Provisions of Sec. 6.12 shall apply for design of composite concrete flexural members defined as precast concrete, cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

6.12.1.2 All provisions of the Code shall apply to composite concrete flexural members, except as specifically modified in Sec. 6.12.

6.12.2 General

6.12.2.1 The use of an entire composite member or portions thereof for resisting shear and moment shall be permitted.

- 6.12.2.2 Individual elements shall be investigated for all critical stages of loading.
- 6.12.2.3 If the specified strength, unit weight, or other properties of the various elements are different, properties of the individual elements or the most critical values shall be used in design.
- 6.12.2.4 In strength computations of composite members, no distinction shall be made between shored and unshored members.
- 6.12.2.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.
- 6.12.2.6 Reinforcement shall be provided as required to minimize cracking and to prevent separation of individual elements of composite members.
- 6.12.2.7 Composite members shall meet requirements for control of deflections in accordance with 6.2.5.4.

6.12.3 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.

6.12.4 Vertical shear strength

- 6.12.4.1 Where an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Sec. 6.4 as for a monolithically cast member of the same cross-sectional shape.
- 6.12.4.2 Shear reinforcement shall be fully anchored into interconnected elements in accordance with 8.2.10.
- 6.12.4.3 Extended and anchored shear reinforcement shall be permitted to be included as ties for horizontal shear.

6.12.5 Horizontal shear strength

- 6.12.5.1 In a composite member, full transfer of horizontal shear forces shall be ensured at contact surfaces of interconnected elements.
- 6.12.5.2 For the provisions of 6.12.5, d shall be taken as the distance from extreme compression fiber for entire composite section to centroid of longitudinal tension reinforcement, if any.
- 6.12.5.3 Unless calculated in accordance with 6.12.5.4, design of cross sections subject to horizontal shear shall be based on

$$V_u \leq \phi V_{nh} \quad (6.12.1)$$

where V_{nh} is nominal horizontal shear strength in accordance with 6.12.5.3.1 through 6.12.5.3.4.

- 6.12.5.3.1 Where contact surfaces are clean, free of laitance, and intentionally roughened, V_{nh} shall not be taken greater than $0.55b_v d$.
- 6.12.5.3.2 Where minimum ties are provided in accordance with 6.12.6, and contact surfaces are clean and free of laitance, but not intentionally roughened, V_{nh} shall not be taken greater than $0.55b_v d$.
- 6.12.5.3.3 Where ties are provided in accordance with 6.12.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 6 mm, V_{nh} shall be taken equal to $(1.8 + 0.6\rho_v f_y)\lambda b_v d$, but not greater than $3.5b_v d$. Values for λ in 6.4.5.4.3 shall apply and ρ_v is $A_v/(b_v s)$.

- 6.12.5.3.4 Where V_u at section considered exceeds $\phi(3.5b_v d)$, design for horizontal shear shall be in accordance with 6.4.5.4.
- 6.12.5.4 As an alternative to 6.12.5.3, horizontal shear shall be permitted to be determined by computing the actual change in compressive or tensile force in any segment, and provisions shall be made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force V_u shall not exceed horizontal shear strength ϕV_{nh} as given in 6.12.5.3.1 through 6.12.5.3.4, where area of contact surface shall be substituted for $b_v d$.
- 6.12.5.4.1 Where ties provided to resist horizontal shear are designed to satisfy 6.12.5.4, the tie area to tie spacing ratio along the member shall approximately reflect the distribution of shear forces in the member.
- 6.12.5.5 Where tension exists across any contact surface between interconnected elements, shear transfer by contact shall be permitted only when minimum ties are provided in accordance with 6.12.6.
- 6.12.6 Ties for horizontal shear**
- 6.12.6.1 Where ties are provided to transfer horizontal shear, tie area shall not be less than that required by 6.4.3.5.3, and tie spacing shall not exceed four times the least dimension of supported element, nor exceed 600 mm.
- 6.12.6.2 Ties for horizontal shear shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire reinforcement.
- 6.12.6.3 All ties shall be fully anchored into interconnected elements in accordance with 8.2.10.

MASONRY STRUCTURES

7.1 INTRODUCTION

7.1.1 Scope

This chapter of the Code covers the design, construction and quality control of masonry structures.

7.1.2 Symbols and Notation

The following units shall be generally implicit in this chapter for the corresponding quantities:

Lengths	mm
Areas	mm ²
Moment of inertia	mm ⁴
Force	N
Moment, torsion	N mm
Stress, strength	N/mm ²

a = depth of equivalent rectangular stress block for strength design

A_b = cross-sectional area of anchor bolt

A_e = effective area of masonry

A_g = gross area of wall

A_{mv} = net area of masonry section bounded by wall thickness and length of section in the direction of shear force considered

A_p = area of tension (pullout) cone of an embedded anchor bolt projected into the surface of masonry

A_s = effective cross-sectional area of reinforcement in a flexural member

A_v = area of steel required for shear reinforcement perpendicular to the longitudinal reinforcement

A'_s = effective cross-sectional area of compression reinforcement in a flexural member

b = effective width of rectangular member or width of flange for T and I section

b_t = computed tension force on anchor bolt

b_v = allowable shear force on anchor bolt

b_w = width of web in T and I member

B_t = allowable tension force on anchor bolt

B_v = computed shear force on anchor bolt

c = distance from the neutral axis to extreme fibre

C_d = masonry shear strength coefficient

d = distance from the compression face of a flexural member to the centroid of longitudinal tensile reinforcement

d_b = diameter of the reinforcing bar, diameter of bolt

e = eccentricity of P_u

e_{mu} = maximum usable compressive strain of masonry

E_m	=	modulus of elasticity of masonry
E_s	=	modulus of elasticity of steel
f_a	=	computed axial compressive stress due to design axial load
f_b	=	computed flexural stress in the extreme fibre due to design bending load only
f_{md}	=	computed compressive stress in masonry due to dead load only
f_r	=	modulus of rupture
f_s	=	computed stress in reinforcement due to design load
f_y	=	tensile yield stress of reinforcement
f_v	=	computed shear stress due to design load
f'_m	=	specified compressive strength of masonry at the age of 28 days
F	=	loads due to weight and pressure of fluids or related moments and forces
F_a	=	allowable average axial compressive stress for centroidally applied axial load only
F_b	=	allowable flexural compressive stress if members were carrying bending load only
F_{br}	=	allowable bearing stress
F_s	=	allowable stress in reinforcement
F_{sc}	=	allowable compressive stress in column reinforcement
F_t	=	allowable flexural tensile stress in masonry
F_v	=	allowable shear stress in masonry
G	=	shear modulus of masonry
h	=	height of wall between points of support
h'	=	effective height of a wall or column
H	=	actual height between lateral supports
H'	=	height of opening
I	=	moment of inertia about the neutral axis of the cross-sectional area
I_g, I_{cr}	=	gross, cracked moment of inertia of the wall cross-section
j	=	ratio or distance between centroid of flexural compressive force and centroid of tensile forces to
		depth, d
k	=	ratio of depth of the compression zone in flexural member to depth, d ; stiffening coefficient
ℓ	=	length of a wall or segment
ℓ_b	=	embedment depth of anchor bolt
ℓ_{be}	=	anchor bolt edge distance, the least length measured from the edge of masonry to the surface of
		the anchor bolt
ℓ_d	=	required development length of reinforcement
L	=	actual length of wall
M	=	design moment
M_c	=	moment capacity of the compression steel in a flexural member about the centroid of the tensile
		force
M_{cr}	=	cracking moment strength of the masonry wall
M_m	=	the moment of the compressive force in the masonry about the centroid of the tensile force in the reinforcement
M_n	=	nominal moment strength of the masonry wall

- M_s = the moment of the tensile force in the reinforcement about the centroid of the compressive force in the masonry
 M_{ser} = service moment at the mid-height of the panel, including P-Delta effects
 M_u = factored moment
 n = modular ratio = E_s/E_m
 P = design axial load
 P_a = allowable centroidal axial load for reinforced masonry columns
 P_b = nominal balanced design axial strength
 P_f = load from tributary floor or roof area
 P_o = nominal axial load strength with bending
 P_u = factored axial load
 P_{uf} = factored load from tributary floor or roof loads
 P_{uw} = factored weight of the wall tributary to the section under consideration
 P_w = weight of the wall tributary to the section under consideration
 r_b = ratio of the area of bars cut off to the total area of bars at the section
 s = spacing of stirrups or bent bars in a direction parallel to that of the main reinforcement
 S = section modulus
 t = effective thickness of a wythe, wall or column
 u = bond stress per unit of surface area of bar
 V = total design shear force
 V_n = nominal shear strength
 V_m = nominal shear strength provided by masonry
 V_s = nominal shear strength provided by shear reinforcement
 Δ_u = horizontal deflection at mid-height under factored load; P-Delta effects shall be included in deflection calculation
 ρ = steel ratio = A_s/bd
 ρ_n = ratio of distributed shear reinforcement on a plane perpendicular to the plane of A_{mv}
 Σ_o = sum of the perimeters of all the longitudinal reinforcement
 ϕ = strength reduction factor.

7.1.3 Definitions

For the purpose of this chapter, the following definitions shall be applicable.

BED BLOCK : A block bedded on a wall, column or pier to disperse a concentrated load on a masonry element.

BED JOINT : A horizontal mortar joint upon which masonry units are placed.

BOND : Arrangement of masonry units in successive courses to tie the masonry together both longitudinally and transversely; the arrangement is usually worked out to ensure that no vertical joint of one course is exactly over the one in the next course above or below it and there is maximum possible amount of lap.

BOND BEAM : A horizontal grouted element within masonry in which reinforcement is embedded.

BUTTRESS : A pier of masonry built as an integral part of wall and projecting from either or both surfaces, decreasing in cross-sectional area from base to top and conforming to the requirement of Sec 4.3.3(c) (ii).

CAVITY WALL : A wall comprising two limbs each built-up as single or multi-wythe units and separated by a 50-115 mm wide cavity. The limbs are tied together by metal ties or bonding units for structural integrity.

CELL : A void space having a gross cross-sectional area greater than 1000 mm².

COLUMN: An isolated vertical load bearing member the width of which does not exceed three times the thickness.

CROSS JOINT: A vertical joint normal to the face of the wall.

CROSS-SECTIONAL AREA OF MASONRY UNIT: Net cross-sectional area of masonry unit is the gross cross-sectional area minus the area of cellular space.

CURTAIN WALL: A non load bearing self supporting wall subject to transverse lateral loads, and laterally supported by vertical or horizontal structural member where necessary.

DIMENSIONS :

Actual dimensions - the measured dimensions of a designated item; such as a designated masonry unit or wall used in the structures. The actual dimension shall not vary from the specified dimension by more than the amount allowed in the appropriate standard mentioned in Sec 2.2.4 of Part 5.

Nominal dimensions; specified dimensions plus the thickness of the joint with which the unit is laid.

Specified dimensions - the dimensions specified for the manufacture or construction of masonry, masonry units, joints or any other components of a structure. Unless otherwise stated, all calculations shall be made using or based on specified dimensions.

FACED WALL: A wall in which facing and backing of two different materials are bonded together to ensure common action under load.

GROUT : A mixture of cementitious materials and aggregate to which water is added such that the mixture will flow without segregation of the constituents.

GROUTED MASONRY :

Grouted hollow-unit masonry - that form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.

Grouted multi-wythe masonry - that form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

HOLLOW UNIT: A masonry unit of which net cross-sectional area in any plane parallel to the bearing surface is less than 75 per cent of its gross cross-sectional area measured in the same plane.

JAMB : Side of an opening in wall.

JOINTS :

Bed joints ; the mortar joint that is horizontal at the time the masonry units are placed.

Collar joint ; the vertical, longitudinal, mortar or grouted joints.

Head joint ; the mortar joint having a vertical transverse plane.

LATERAL SUPPORT : A support which enables a masonry element to resist lateral load and/or restrains lateral deflection of a masonry element at the point of support.

LIMB : Inner or outer portion of a cavity wall.

LOAD BEARING WALL : A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load.

MASONRY : An assemblage of masonry units properly bonded together with mortar.

MASONRY UNIT : Individual units, such as brick, tile, stone or concrete block, which are bonded together with mortar to form a masonry element such as walls, columns, piers, buttress, etc.

PANEL WALL : An exterior non load bearing wall in framed structure, supported at each storey but subject to lateral loads.

PARTITION WALL : An interior non load bearing wall, one storey or part storey in height.

PIER : A projection from either or both sides of a wall forming an integral part of the wall and conforming to the requirement of Sec 4.4.3.3.c(ii).

PILASTER : A thickened section forming integral part of a wall placed at intervals along the wall, to increase the stiffness of the wall or to carry a vertical concentrated load. Thickness of a pier is the overall thickness including the thickness of the wall or, when bounded into a limb of cavity wall, the thickness obtained by treating that limb as an independent wall.

PRISM : An assemblage of masonry units bonded by mortar with or without grout used as a test specimen for determining properties of masonry.

REINFORCED MASONRY : The masonry construction, in which reinforcement acting in conjunction with the masonry is used to resist forces and is designed in accordance with Sec 4.6.

SHEAR WALL : A load bearing wall designed to carry horizontal forces acting in its own plane with or without vertical imposed loads.

SOLID UNIT : A masonry unit whose net cross-sectional area in any plane parallel to the bearing surface is 75 per cent or more of the gross cross-sectional area in the same plane.

STACK BOND : A bond in bearing and nonbearing walls, except veneered walls, in which less than 75 per cent of the units in any transverse vertical plane lap the ends of the units below a distance less than one-half the height of the unit, or less than one-fourth the length of the unit.

VENEERED WALL : A wall in which the facing is attached to the backing but not so bonded as to result in a common action under load.

WALL JOINT : A vertical joint parallel to the face of the wall.

WALL TIE : A metal fastener which connects wythes of masonry to each other or to other materials.

WYTHER : Portion of a wall which is one masonry unit in thickness.

7.2 Materials

7.2.1 General

All materials used in masonry construction shall conform to the requirements specified in Part 5 of this Code. If no requirements are specified for a material, quality shall be based on generally accepted good practice, subject to the approval of the building official.

7.2.2 Masonry Units

The following types of masonry units which conform to the standards mentioned in Sec 2.2.4 of Part 5 may be used in masonry construction:

- (a) Common building clay bricks
- (b) Burnt clay hollow bricks
- (c) Burnt clay facing bricks
- (d) Hollow concrete blocks

Other types of masonry units conforming to Sec 2.2.4 of Part 5 may also be used.

7.2.3 Mortar and Grout

Mortar and grout for masonry construction shall conform to the requirements specified in Part 5. Mix proportions and compressive strength of some commonly used mortars are given in Table 6.7.1.

7.3 Allowable STRESSES

7.3.1 General

Stresses in masonry shall not exceed the values given in this section. All allowable stresses for working stress design may be increased one third when considering wind or earthquake forces either acting alone or combined with vertical loads. No increase shall be allowed for vertical loads acting alone.

7.3.2 Specified Compressive Strength of Masonry, f'_m

The allowable stresses for masonry construction shall be based on the value of f'_m as determined by Sec 7.3.3 below.

Table 6.7.1: Mix Proportion and Strength of Commonly used Mortars

Grade of Mortar	Mix Proportion by Volume 1,2		Minimum Compressive Strength at 28 days, N/mm ²
	Cement	Sand	
M1		3	10
M2		4	7.5
M3	1	5	5
M4		6	3
M5		7	2
M6		8	1

1. Sand and cement shall be measured in loose volume and sand shall be well graded with a minimum F.M. of 1.2.

2. Lime to a maximum of $\frac{1}{4}$ th part by volume of cement may be used to increase workability.

7.3.3 Compliance with f'_m

Compliance with the requirements for the specified compressive strength of masonry, f'_m shall be in accordance with the following:

7.3.3.1 Masonry Prism Testing : The compressive strength of masonry based on tests at 28 days in accordance with "Standard Test Method for Compressive Strength of Masonry Prisms", (ASTM E447) for each set of prisms shall equal or exceed f'_m . Verification by masonry prism testing shall meet the following :

- Testing Prior to Construction: A set of five masonry prisms shall be built and tested in accordance with ASTM E447 prior to the start of construction. Materials used for prisms shall be same as used in the project. Prisms shall be constructed under the observation of the engineer or an approved agency and tested by an approved agency.
- Testing During Construction: When full allowable stresses are used in design, a set of three prisms shall be built and tested during construction in accordance with (ASTM E447) for each 500 square meters of wall area, but not less than one set of three masonry prisms for any project. No testing during construction shall be required when 50% of the allowable stresses are used in design.

7.3.4 Quality Control

Quality control shall include, but not be limited to assure that:

- Masonry units, reinforcement, cement, lime, aggregate and all other materials meet the requirements of the applicable standard of quality and that they are properly stored and prepared for use.
- Mortar and grout are properly mixed using specified proportions of ingredients. The method of measuring materials for mortar and grout shall be such that proportions of materials are controlled.
- Construction details, procedures and workmanship are in accordance with the plans and specification.
- Placement, splices and bar diameters are in accordance with the provisions of this chapter and the plans and specifications.

7.3.5 Allowable Stresses in Masonry

When the quality control provisions specified in Sec 7.3.4 above do not include requirements for special inspection, the allowable design stresses in this section shall be reduced by 50 per cent.

- Compressive Stress, Axial

- i) Unreinforced masonry walls, columns and reinforced masonry wall

$$F_a = \frac{f'_m}{5} \left[1 - \left(\frac{h'}{42t} \right)^3 \right] \quad (7.3.1)$$

- ii) Reinforced masonry columns

$$F_a = \left(\frac{f'_m}{5} + \frac{A_s}{1.5A_e} F_{sc} \right) \left[1 - \left(\frac{h'}{42t} \right)^3 \right] \quad (7.3.2)$$

- (b) Compressive Stress, Flexural

$$F_b = 0.33f'_m \leq 10 \text{ N/mm}^2 \quad (7.3.3)$$

- (c) Tensile Stress for Walls, Flexure

The allowable tensile stress for walls in flexure of masonry structures without tensile reinforcement using mortar Type M₁ or M₂ shall not exceed the values specified in Tables 6.7.2 and 6.7.3. For Type M₃ and M₄ mortar, the value shall be reduced by 25 per cent.

No tension is allowed across head joints in stack bond masonry. Values for tension normal to head joints are for running bond. These values shall not be used for horizontal flexural members such as beams, girders or lintels.

Table 6.7.2 : Flexural Tension, F_t

Masonry	Normal to Bed Joints	Normal to Head Joints
	N/mm ²	N/mm ²
Solid Units	0.20	0.40
Hollow Units	0.12	0.25

Table 6.7.3 : Tension Normal to Head Joints, F_t

Masonry	Clay Units	Concrete Units
	N/mm ²	N/mm ²
Solid Units	0.35	0.40
Hollow Units	0.22	0.25

- (d) Reinforcing Bond Stress, u

Plain Bars 0.30 N/mm²

Deformed Bars 1.0 N/mm²

- (e) Shear Stress for Flexural Members, F_v

- i) When no shear reinforcement is used

$$F_v = 0.083\sqrt{f'_m} \leq 0.25 \text{ N/mm}^2 \quad (7.3.4)$$

- ii) When shear reinforcement is designed to take entire shear force

$$F_v = 0.25\sqrt{f'_m} \leq 0.75 \text{ N/mm}^2 \quad (7.3.5)$$

- (f) Shear Stress for Shear Walls, F_v

- i) Unreinforced masonry

For clay units:

$$F_v = 0.025\sqrt{f'_m} \leq 0.40 \quad \text{N/mm}^2 \quad (7.3.6)$$

For concrete units :

M₁ or M₂ Mortar 0.20 N/mm²

M₃ Mortar 0.12 N/mm²

ii) The allowable shear stress for reinforced masonry shear walls shall be according to Table 6.7.4.

7.3.6 Allowable Stresses in Reinforcement

(a) Tensile Stress

i) Deformed bars,

$$F_s = 0.5 f_y \leq 165 \quad \text{N/mm}^2 \quad (7.3.7)$$

ii) Ties, anchors and plain bars,

$$F_s = 0.4 f_y \leq 135 \quad \text{N/mm}^2 \quad (7.3.8)$$

Table 6.7.4: Allowable Shear Stress for Reinforced Masonry Shear Walls, F_v

	M/Vd	$F_v, \text{N/mm}^2$	Maximum Allowable N/mm ²
Masonry taking all shear	< 1	$\frac{1}{36} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m}$	$\left(0.4 - 0.2 \frac{M}{Vd} \right)$
	≥ 1	$0.083 \sqrt{f'_m}$	0.17
Reinforce- ment taking all shear	< 1	$\frac{1}{24} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m}$	$\left(0.6 - 0.2 \frac{M}{Vd} \right)$
	≥ 1	$0.125 \sqrt{f'_m}$	0.37

(b) Compressive Stress

i) Deformed bars in columns and shear walls,

$$F_{sc} = 0.4 f_y \leq 165 \quad \text{N/mm}^2 \quad (7.3.9)$$

ii) Deformed bars in flexural members

$$F_{sc} = 0.5 f_y \leq 165 \quad \text{N/mm}^2 \quad (7.3.10)$$

7.3.7 Combined Compressive Stress

Members subject to combined axial and flexural stresses shall be designed in accordance with accepted principles of mechanics or in accordance with the following formula:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \quad (7.3.11)$$

7.3.8 Modulus of Elasticity

The modulus of elasticity of masonry shall be determined by the secant method. The slope of the line connecting the points $0.05 f'_m$ and $0.33 f'_m$ on the stress-strain curve shall be taken as the modulus of elasticity

of masonry. If required, actual values shall be established by tests. These values are not to be reduced by 50 per cent as specified in Sec 7.3.5(a).

(a) Modulus of Elasticity for Masonry

$$E_m = 750 f'_m \leq 15,000 \quad \text{N/mm}^2 \quad (7.3.12)$$

(b) Modulus of Elasticity for Steel

$$E_s = 200,000 \quad \text{N/mm}^2 \quad (7.3.13)$$

(c) Shear Modulus of Masonry

$$G = 0.4 E_m \quad (7.3.14)$$

7.3.9 Shear and Tension on Embedded Anchor Bolts

7.3.9.1 Allowable loads and placement requirements for anchor bolts shall be in accordance with the following:

- (a) Bent bar anchor bolts shall have a hook with a 90 degree bend with an inside diameter of 3db plus an extension of 1.5db at the free end.
- (b) Headed anchor bolts shall have a standard bolt head.
- (c) Plate anchor bolts shall have a plate welded to the shank to provide anchorage equivalent to headed anchor bolts.

7.3.9.2 The effective embedment length, ℓ_b for bent bar anchors shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter. For plate or headed anchor bolts ℓ_b shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the plate or head of the anchorage. All bolts shall be grouted in place with at least 25 mm of grout between the bolt and the masonry except that 6 mm diameter bolts may be placed in bed joints which are at least twice as thick as the diameter of the bolt.

7.3.9.3 Allowable Shear Force

Allowable loads in shear shall be according to Table 6.7.5 or lesser of the value obtained from the following formulae:

$$B_v = 1070 (f'_m A_b)^{1/4} \quad (7.3.15)$$

$$B_v = 0.12 A_b f_y \quad (7.3.16)$$

When the distance ℓ_b is less than 12 db, the value of B_v in Eq (7.3.15) shall be reduced to zero at a distance ℓ_b be equal to 40 mm. Where adjacent anchors are spaced closer than 8db, the allowable shear of the adjacent anchors determined by Eq (7.3.15) shall be reduced by interpolation to 0.75 times the allowable shear value at a centre to centre spacing of 4 db.

Table 6.7.5 : Allowable Shear, B_v for Embedded Anchor Bolts for Masonry, kN*

Bent Bar Anchor Bolt Diameter, mm							
f'_m							
N/mm ²	10	12	16	20	22	25	28
10	2.0	3.7	5.9	7.9	8.5	9.1	9.6
12	2.0	3.7	5.9	8.2	8.3	9.5	10.1
13	2.0	3.7	5.9	8.5	9.2	9.8	10.4
17	2.0	3.7	5.9	8.5	9.7	10.3	11.0

Bent Bar Anchor Bolt Diameter, mm							
f'_m							
N/mm ²	10	12	16	20	22	25	28
20	2.0	3.7	5.9	8.5	10.1	10.8	11.5
27	2.0	3.7	5.9	8.5	10.9	11.6	12.3

* Values are for bolts of at least ASTM A307 quality. Bolts shall be those specified in Sec 4.3.9.1.

7.3.9.4 Allowable Tension

Allowable tension shall be the lesser value selected from Table 6.7.6 and Table 6.7.7 or shall be determined from lesser of the values obtained from the following formulae:

$$B_t = 0.04A_p \sqrt{f'_m} \quad (7.3.17)$$

$$B_t = 0.2A_b f_y \quad (7.3.18)$$

The area A_p shall be the lesser of the area obtained from Eq (7.3.17) and (7.3.18) and where the projected areas of adjacent anchor bolts overlap, A_p of each anchor bolt shall be reduced by 50 per cent of the overlapping area.

$$A_p = \pi \ell_b^2 \quad (7.3.19)$$

$$A_p = \pi \ell_{be}^2 \quad (7.3.20)$$

Table 6.7.6: Allowable Tension, B_t for Embedded Anchor Bolts for Masonry, kN ^{1, 2}

Embedment Length, ℓ_b , or Edge Distance, ℓ_{be} , mm							
f'_m							
N/mm ²	50	75	100	125	150	200	250
10	1.0	2.4	4.3	6.7	9.7	17.3	27.0
12	1.2	2.6	4.7	7.4	10.6	18.9	29.6
13	1.2	2.8	5.0	7.8	11.2	20.0	31.2
17	1.3	3.1	5.6	8.7	12.6	22.4	35.0
20	1.5	3.4	6.7	9.5	13.8	24.5	38.2
27	1.7	3.9	7.0	11.0	15.9	28.3	44.1

¹ The allowable tension values are based on compressive strength of masonry assemblages. Where yield strength of anchor bolt steel governs, the allowable tension is given in Table 6.7.7.

² Values are for bolts of at least ASTM A307 quality. Bolts shall be those specified in Sec 7.3.9.1.

Table 6.7.7 : Allowable Tension, B_t for Embedded Anchor Bolts for Masonry, kN ¹

Bent Bar Anchor Bolt Diameter, mm							
6	10	12	16	20	22	25	28
1.5	3.5	6.2	9.8	14.1	19.2	25.1	31.8

¹ Values are for bolts of at least ASTM A307 quality. Bolts shall be those specified in Sec 7.3.9.

7.3.9.5 Combined Shear and Tension

Anchor bolts subjected to combined shear and tension shall be designed in accordance with the formula given below:

$$\frac{b_t}{B_t} + \frac{b_v}{B_v} \leq 1.00 \quad (7.3.22)$$

7.3.9.6 Minimum Edge Distance, ℓ_{be}

The minimum value of ℓ_{be} be measured from the edge of the masonry parallel to the anchor bolt to the surface of the anchor bolt shall be 40 mm.

7.3.9.7 Minimum Embedment Depth, ℓ_b

The minimum embedment depth ℓ_b shall be $4d_b$ but not less than 50 mm.

7.3.9.8 Minimum Spacing Between Bolts

The minimum centre to centre spacing between anchors shall be $4d_b$.

7.3.10 Load Test

For load test, the member shall be subject to a superimposed load equal to twice the design live load plus one-half of the dead load. This load shall be maintained for a period of 24 hours. If, during the test or upon removal of the load, the member 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 possible, a lower rating shall be established. A flexural member shall be considered to have passed the test if the maximum deflection at the end of the 24 hour period neither exceeds 0.005ℓ nor $0.00025\ell^2/t$ and the beam and slabs show a recovery of at least 75 per cent of the observed deflection within 24 hours after removal of the load.

7.3.11 Reuse of Masonry Units

Masonry units may be reused when clean, unbroken and conforms to the requirements of Part 5. All structural properties of masonry of reclaimed units, especially adhesion bond, shall be determined by approved test. The allowable working stress shall not exceed 50 per cent of that permitted for new masonry units of the same properties.

7.4 BASIC DESIGN REQUIREMENTS

7.4.1 General

Masonry structures shall be designed according to the provisions of this section. The required design strengths of masonry materials and any special requirements shall be specified in the plan submitted for approval.

7.4.2 Design Considerations

7.4.2.1 Masonry structures shall be designed based on working stress and linear stress-strain distribution. Requirements for working stress design of unreinforced and reinforced masonry structures are provided in Sec 4.5 and 4.6 respectively. In lieu of the working stress design method, slender walls and shear walls may be designed by the strength design method specified in Sec 7.7.

The structure shall be proportioned such that eccentricity of loading on the members is as small as possible. Eccentric loading shall preferably be avoided by providing:

- (a) adequate bearing of floor/roof on the walls
- (b) adequate stiffness in slabs, and
- (c) fixity at the supports.

7.4.2.2 Effective Height

- (a) Wall : The effective height of a wall shall be taken as the clear height between the lateral supports at top and bottom in a direction normal to the axis considered. For members not supported at the top normal to the axis considered, the effective height is twice the height of the member above the support. Effective height less than the clear height may be used if justified.
- (b) Column : Effective height of the column shall be taken as actual height for the direction it is laterally supported and twice the actual height for the direction it is not laterally supported at the top normal to the axis considered.
- (c) Opening in Wall : When openings occur in a wall such that masonry between the openings is by definition a column, effective height of masonry between the openings shall be obtained as follows:
 - i) When wall has full restraint at the top, effective height for the direction perpendicular to the plane of wall equals $0.75H$ plus $0.25H'$, where H is the distance between supports and H' is the height of the taller opening; and effective height for the direction parallel to the wall equals H .
 - ii) When wall has partial restraint at the top and bottom, effective height for the direction perpendicular to the plane of wall equals H when height of neither opening exceeds $0.5H$ and it is equal to $2H$ when height of any opening exceeds $0.5H$; and effective height for the direction parallel to the plane of the wall equals $2H$.

7.4.2.3 Effective Length

Effective length of a wall for different support conditions shall be as given in Table 6.7.8.

7.4.2.4 Effective Thickness

The effective thickness of walls and columns for use in the calculation of slenderness ratio, shall be defined as follows:

- (a) Solid Walls: The effective thickness of solid walls, faced walls or grouted walls shall be the specified thickness of the wall.
- (b) Solid Walls with Raked Mortar Joints: The effective thickness of solid walls with raked mortar joints shall be the minimum thickness measured at the joint.
- (c) Cavity Walls: When both limbs of a cavity wall are axially loaded, each limb shall be considered independently and the effective thickness of each limb shall be determined as in (a) or (b) above. If one of the limbs is axially loaded, the effective thickness of the cavity wall shall be taken as the square root of the sum of the squares of the effective thicknesses of the limbs.
- (d) Walls Stiffened by Pilasters: When solid or cavity walls are stiffened by pilasters at intervals, the effective thickness to be used for the calculation of h'/t ratio shall be determined as follows:
 - i) Solid Walls: For stiffened solid walls the effective thickness shall be the specified thickness multiplied by the stiffening coefficient, k , values of which are given below:

ℓ_p/w_p	Stiffening Coefficient, k^*		
	t_p/t_w		
	1	2	3
6	1.0	1.4	2.0
8	1.0	1.3	1.7
10	1.0	1.2	1.4
15	1.0	1.1	1.2
20 or more	1.0	1.0	1.0

* Linear interpolation is permitted for obtaining intermediate values of k

where, ℓ_p = centre to centre spacing of pilasters

t_p = thickness of pilaster including the wall

t_w = specified thickness of main wall

w_p = width of pilaster in the direction of wall

- ii) Cavity Walls: When one or both limbs of a cavity wall are adequately bonded into pilasters at intervals, the effective thickness of each limb shall be determined separately as in (a), (b) or d(i) above and the effective thickness of the stiffened cavity wall shall be determined in accordance with (c) above.

Where slenderness ratio of the wall is based on the effective length, the effective thickness shall be the same as that without pilasters.

- (e) Columns: The effective thickness for rectangular columns in the direction considered is the actual thickness provided in that direction. The effective thickness for nonrectangular columns is the thickness of a square column with the same moment of inertia about its axis as that about the axis considered in the actual column.

Table 6.7.8: Effective Length of Walls

Support Condition	Effective Length
Where a wall is continuous and is supported by cross wall and there is no opening within a distance of $H/8$ from the face of cross wall, Or Where a wall is continuous and is supported by pier/buttresses conforming to Sec 7.4.3.3 (c) (ii).	0.8L
Where a wall is supported by cross wall at one end and continuous with cross wall at other end, Or Where a wall is supported by pier/buttresses at one end and continuous with pier/buttresses at other end conforming to Sec 7.4.3.3 (c) (ii).	0.9L
Where a wall is supported at each end by cross wall, Or Where a wall is supported at each end by pier/buttresses conforming to Sec 7.4.3.3 (c) (ii).	1.0L
Where a wall is free at one end and continuous with a cross wall at the other end, Or Where a wall is free at one end and continuous with a pier/buttresses at the other end conforming to Sec 7.4.3.3 (c) (ii).	1.5L
Where a wall is free at one end and supported at the other end by a cross wall, Or Where a wall is free at one end and supported at the other end by a pier/buttresses conforming to Sec 7.4.3.3 (c) (ii).	2.0L

7.4.2.5 Slenderness Ratio

- (a) Walls: For a wall, slenderness ratio shall be the ratio of effective height to effective thickness or effective length to effective thickness whichever less is. In case of a load bearing wall, slenderness ratio shall not exceed 20.
- (b) Column: For a column, slenderness ratio shall be taken to be the greater of the ratio of effective heights to the respective effective thickness in the two principal directions. Slenderness ratio for a load bearing column shall not exceed 12.

7.4.2.6 Effective Area

The effective cross-sectional area shall be based on the minimum bedded area of the hollow units, or the gross area of solid units plus any grouted area. If hollow units are used perpendicular to the direction of stress, the effective area shall be lesser of the minimum bedded area or the minimum cross-sectional area. If bed joints are raked, the effective area shall be correspondingly reduced. Effective areas for cavity walls shall be that of the loaded wythes.

7.4.2.7 Flexural Resistance of Cavity Walls

For computing the flexural resistance, lateral loads perpendicular to the plane of the wall shall be distributed to the wythes according to their respective flexural rigidities.

7.4.2.8 Effective Width of Intersecting Walls

Where a shear wall is anchored to an intersecting wall or walls, the width of the overhanging flange formed by the intersected walls on either side of the shear wall shall not exceed 6 times the thickness of the intersected wall. Limits of the effective flange may be waived if justified. Only the effective area of the wall parallel to the shear forces may be assumed to carry horizontal shear.

7.4.3 Supports

7.4.3.1 Vertical Support

Structural members providing vertical support of masonry shall provide a bearing surface on which the initial bed joint shall not be less than 6 mm or more than 25 mm and shall be of noncombustible materials, except where masonry is a nonstructural decorative feature or wearing surface.

7.4.3.2 Vertical Deflection

Elements supporting masonry shall be designed so that their vertical deflection does not exceed 1/600 of the clear span under total loads. Lintels shall be supported on each end such that allowable stresses in the supporting masonry are not exceeded. The minimum bearing length shall be 100 mm.

7.4.3.3 Lateral Support

- (a) Lateral support of masonry may be provided by cross walls, columns, piers, counter forts or buttresses when spanning horizontally or by floors, beams or roofs when spanning vertically.
- (b) Lateral supports for a masonry element such as load bearing wall or column shall be provided to
 - i) limit the slenderness of a masonry element so as to prevent or reduce possibility of buckling of the member due to vertical loads; and
 - ii) resist the horizontal components of forces so as to ensure stability of a structure against overturning.
- (c) From consideration of slenderness (i.e. requirement b(i) above), masonry elements may be considered to be laterally supported if
 - i) in case of a wall, where slenderness ratio is based on effective height, floor/roof slab (or beams and slab) irrespective of the direction of span, bears on the supported wall as well as cross walls, to the extent of at least 100 mm;

- ii) in case of a wall, when slenderness ratio is based on its effective length, a cross wall/pier/buttress of thickness equal to or more than half the thickness of the supported wall or 125 mm, whichever is more and average length equal to or more than one-fifth of the height of the wall, is built at right angle to the wall and properly bonded;
- iii) in case of a column, an RC or timber beam/R S joist/roof truss, is supported on the column. In this case, the column will not be considered to be laterally supported in the direction at right angle to it; and
- iv) in case of a column, an RC beam forming a part of beam and slab construction, is supported on the column, and the slab adequately bears on stiffening walls. This construction will provide lateral support to the column, in the direction of both horizontal axes.

7.4.4 Stability

A wall or column subject to vertical and lateral loads may be considered to provide adequate lateral support from consideration of stability, if the construction providing the support is capable of resisting the following forces:

- (a) Simple static reactions at the point of lateral support to all the lateral loads; plus
- (b) A lateral load equal to 2.5% of the total vertical load that the wall or column is designated to carry at the point of lateral support.

7.4.4.1 In case of load bearing buildings up to five storeys, stability requirements may be considered to have been satisfied if the following conditions are met.

- (a) Height to width ratio of building does not exceed 2.
- (b) Cross walls acting as stiffening walls continuous from outer wall to outer wall or outer wall to a load bearing inner wall, and of thickness and spacing as given in Table 6.7.9 are provided.
Note : If stiffening wall or walls that are in a line, are interrupted by openings, length of solid wall or walls in the zone of the wall that is to be stiffened shall be at least one-fifth of the height of the opening.
- (c) Floors and roof either bear on cross walls or are properly anchored to those walls such that all lateral loads are safely transmitted to those walls and through them to the foundation.
- (d) Cross walls are built jointly with the bearing walls and jointly mortared, or interconnected by toothing.

Note : Cross walls may be anchored to walls to be supported by ties of noncorrosive metal of minimum section 6 x 35 mm and length 60 mm with ends bent at least 50 mm, maximum vertical spacing of ties being 1.2 m.

Table 6.7.9: Thickness and Spacing of Stiffening Walls

Stiffening Wall *				
Thickness of Load Bearing Wall to be Stiffened (mm)	Storey Height not to Exceed (m)	Thickness not less than		Maximum spacing (m)
		1 to 3 storeys (mm)	4 and 5 storeys (mm)	
100	3.2	100	-	4.5
200	3.2	100	200	6.0
300	3.4	100	200	8.0
above 300	5.0	100	200	8.0

* Storey height and maximum spacing as given are centre to centre dimensions.

- 7.4.4.2 In case of walls exceeding 8.0 m in length, safety and adequacy of lateral supports shall always be checked by structural analysis.
- 7.4.4.3 A trussed roofing may not provide lateral support unless special measures are adopted to brace and anchor the roofing. However, in case of residential and similar buildings of conventional design with trussed roofing having cross walls, it may be assumed that stability requirements are met by the cross walls and structural analysis for stability may be dispensed with.
- 7.4.4.4 In case of walls exceeding 8.0 m in length, safety and adequacy of lateral supports shall always be checked by structural analysis.
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- 7.4.4.6 In case of external walls of basement and plinth, stability requirements of Sec 7.4.4 may be considered to be satisfied if :
- Bricks used in basement and plinth have a minimum crushing strength of 5 N/mm² and mortar used in masonry is of Type M₃ or better,
 - Clear height of ceiling in basement does not exceed 2.6 m,
 - In the zone of action of soil pressure on basement walls, traffic load excluding any surcharge due to adjoining buildings does not exceed 5 kN/m²,
 - Minimum thickness of basement walls is in accordance with Table 6.7.10.

In case there is surcharge on basement walls from adjoining buildings, thickness of basement walls shall be based on structural analysis.

Table 6.7.10: Minimum Thickness of Basement Wall

Minimum Thickness of Basement Wall (Nominal), mm	Height of the Ground above Basement Floor Level with Wall Loading (Permanent Load), m	
	Less than 50 kN/m	More than 50 kN/m
375	2.0	2.5
250	1.4	1.8

7.4.4.7 Free Standing Wall

Free standing walls, subject to wind pressure or seismic forces shall be designed on the basis of permissible tensile stress in masonry or stability consideration. However in Seismic Zones 1 and 2, free standing walls may be proportioned without making any design calculations with the help of Table 6.7.11 provided the mortar used is of type not leaner than M3. For parapet wall see Sec 7.4.9.4.

7.4.5 Structural Continuity

Intersecting structural elements intended to act as a unit shall be anchored together to resist the design forces. Walls shall be anchored together to all floors, roofs or other elements which provide lateral support for the wall. Where floors or roofs are designed to transmit horizontal forces to walls, the anchorages to the walls shall be designed to resist the horizontal forces.

Table 6.7.11: Height to Thickness Ratio of Free Standing Wall

Design Wind Pressure, N/m ²	Height to Thickness Ratio
Up to 300	10
600	7
900	5
1100	4

Note : Height is to be taken from 150 mm below ground level or top of footing/foundation block, whichever is higher, and up to the top edge of the wall.

7.4.5.1 Multi-wythe Walls

All wythes shall be bonded by grout or tied together by corrosion resistant wall ties or joint reinforcement as follows:

- (a) **Wall Ties in Cavity Wall Construction:** Wall ties shall be of sufficient length to engage all wythes. The portion of the wall ties within the wythe shall be completely embedded in mortar or grout. The ends of the wall ties shall be bent to 90 degree angles with an extension not less than 50 mm long. Wall ties not completely embedded in mortar or grout between wythes shall be a single piece with each end engaged in each wythe.

There shall be at least one 6 mm diameter wall tie for each 0.45 m² of wall area. For cavity walls in which the width of the cavity is greater than 75 mm, but not more than 115 mm, at least one 6 mm diameter wall tie for each 0.3 m² of wall area shall be provided.

Ties in alternate courses shall be staggered. The vertical distance between ties shall not exceed 600 mm. The horizontal distance between ties shall not exceed 900 mm.

Additional ties spaced not more than 900 mm apart shall be provided around and within 300 mm of the opening.

Wall ties of different size and spacing may be used if they provide equivalent strength between wythes.

- (b) **Wall Ties for Grouted Multi-wythe Construction:** The two wythes shall be bonded together with at least 6 mm diameter steel wall ties for each 0.20 m² of area. Wall ties of different size and spacing may be used if they provide equivalent strength between wythes.
- (c) **Joint Reinforcement:** Prefabricated joint reinforcement for masonry walls shall have a minimum of one cross wire of at least 3 mm diameter steel for each 0.2 m² of wall area. The vertical spacing of the joint reinforcement shall not exceed 400 mm. The longitudinal wires shall be thoroughly embedded in the bed joint mortar. The joint reinforcement shall engage all wythes.

Where the space between tied wythes is filled with grout or mortar, the allowable stresses and other provisions for masonry bonded walls shall apply. Where the space is not filled, tied walls shall conform to the allowable stress, lateral support, thickness (excluding cavity), height and tie requirements of cavity walls.

7.4.6 Joint Reinforcement and Protection of Ties

The minimum mortar cover between ties or joint reinforcement and any exposed face shall be 15 mm. The thickness of grout or mortar between masonry units and joint reinforcement shall not be less than 6 mm, except that smaller diameter reinforcement or bolts may be placed in bed joints which are at least twice as thick as the diameter of the reinforcement.

7.4.7 Pipes and Conduits

Pipe or conduit shall not be embedded in any masonry so as to reduce the capacity to less than that necessary for required stability or required fire protection, except the following:

- (a) Rigid electrical conduit may be embedded in structural masonry when their location has been detailed on the approved plan.

- (b) Any pipe or conduit may pass vertically or horizontally through any masonry by means of a sleeve at least large enough to pass any hub or coupling on the pipeline. Such sleeves shall not be placed closer than three diameters, centre to centre, nor shall they unduly impair the strength of construction.
- (c) Placement of pipes or conduits in unfilled cores of hollow unit masonry shall not be considered as embedment.

7.4.8 Loads and Load Combination

7.4.8.1 Design Loads

All design loads and other forces to be taken for the design of masonry structures shall conform to Chapter 2, Loads.

7.4.8.2 Load Dispersion

The angle of dispersion of vertical load on walls shall be taken as not more than 30° from the vertical.

7.4.8.3 Distribution of Concentrated Vertical Loads in Walls

The length of wall, laid up in running bond, which may be considered capable of working at the maximum allowable compressive stresses to resist vertical concentrated loads, shall not exceed the centre to centre distance between such loads, nor the width of bearing area plus four times the wall thickness. Concentrated vertical loads shall not be assumed distributed across continuous vertical mortar or control joints unless elements designed to distribute the concentrated vertical loads are employed.

7.4.8.4 Loads on Nonbearing Wall

Masonry walls used as interior partition or as exterior surfaces of building which do not carry vertical loads imposed by other elements of the building shall be designed to carry their own weight plus any superimposed finish and lateral forces. Bonding or anchorage of nonbearing walls shall be adequate to support the walls and to transfer lateral forces to the supporting structures.

7.4.8.5 Load Combinations

Load combination for design of masonry structures shall conform to the requirements of Sec 2.7.5.1.

7.4.9 Minimum Design Dimensions

7.4.9.1 Minimum Thickness of Load Bearing Walls

The nominal thickness of masonry bearing walls in building shall not be less than 250 mm.

Exception:

Stiffened solid masonry bearing walls in one-storey buildings may have a minimum effective thickness of 165 mm when not over 3 m in height, provided that when gable construction is used an additional 1.5 m height may be permitted at the peak of the gable.

7.4.9.2 Variation in Thickness

When a change in thickness due to minimum thickness requirements occurs between floor levels, the greater thickness shall be carried up to the higher floor level.

7.4.9.3 Decrease in Thickness

When walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be constructed between the walls below and the thinner wall above, or special units or construction shall be used to transmit the loads from wythes to the walls below.

7.4.9.4 Parapet Wall

Parapet walls shall be at least 200 mm thick and height shall not exceed 4 times the thickness. The parapet wall shall not be thinner than the wall below.

7.5 DESIGN OF UNREINFORCED MASONRY

7.5.1 General

The requirements of this section are applicable to unreinforced masonry in addition to the requirements of Sec 7.4.

7.5.2 Design of Members Subjected to Axial Compression

The stresses due to compressive forces applied at the centroid of any load bearing wall, column and pilaster may be computed by Eq (7.5.1) below assuming uniform distribution over the effective area.

$$f_a = \frac{P}{A_e} \quad (7.5.1)$$

7.5.3

Design of Members Subjected to Combined Bending and Axial Compression

- (a) Compressive stresses due to combined bending and axial load shall satisfy the requirements of Sec 7.3.5.
- (b) Resultant tensile stress due to combined bending and axial load shall not exceed the allowable flexural tensile stress, F_t as specified in Sec 7.3.

7.5.4 Design of Members Subjected to Flexure

Stresses due to flexure calculated by Eq (7.5.2) below shall not exceed the values given in Sec 7.3.5.

$$f_b = \frac{Mc}{I} \quad (7.5.2)$$

7.5.5 Design of Members Subjected to Shear

Shear calculations in flexural members and shear walls shall be based on Eq (7.5.3) below.

$$f_v = \frac{V}{A_e} \quad (7.5.3)$$

7.5.6 Design of Arches

Geometrical form and the cross-sectional dimensions of masonry arch shall be selected such that the line of thrust at any section of the arch is kept within the middle third of the section of the arch rib. The elastic theory of arches shall be permitted for the analysis of unreinforced masonry arches. All supports of arches shall be capable of developing the required horizontal thrust without suffering unacceptable displacements. Every arch must be designed to resist the stresses due to the following loads:

- (a) Gravity Loads :
 - i. Dead loads shall be placed in conformity with their actual distribution.
 - ii. Live loads shall be positioned to cover entire span or part of the span as necessary to produce the maximum stresses at the crown, springing and all other sections of the arch rib.
- (b) Loads due to temperature change.
- (c) Shrinkage load due to setting and hardening.
- (d) Shortening of arch rib under thrust caused by loads.

7.5.7 Footings and Corbels

The slope of footing and corbelling (measured from the horizontal to the face of the corbelled surface) shall not be less than 60 degrees.

The maximum horizontal projection of corbelling from the plane of the wall shall be such that stress at any section does not exceed the allowable value.

7.6 DESIGN OF REINFORCED MASONRY

7.6.1 General

The requirements of this section are in addition to those specified in Sec 7.4 and are applicable to reinforced masonry. Plain bars larger than 6 mm in diameter shall not be used.

7.6.1.1 Assumptions

The following assumptions shall be applicable for this section.

- (a) Masonry carries no tensile stress.
- (b) Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogeneous material within the range of working stresses.

7.6.2 Design of Members Subjected to Axial Compression

Stresses due to compressive forces applied at the centroid of load bearing wall, column and pilaster may be computed assuming uniform distribution over the effective area. Stress shall be calculated from Eq (7.6.1) below:

$$f_a = \frac{P}{A_e} \quad (7.6.1)$$

7.6.3 Design of Members Subjected to Combined Bending and Axial Compression

Stress due to combined bending and axial loads shall satisfy the requirements of Sec 7.3.5. Columns and walls subjected to bending with or without axial loads shall meet all applicable requirements for flexural design.

The design of walls with an (h'/t) ratio larger than 30 shall be based on forces and moments determined from analysis of 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.

7.6.4 Design of Members Subjected to Shear Force

Shearing stresses in flexural members and shear walls shall be computed by

$$f_v = \frac{V}{bjd} \quad (7.6.2)$$

When the computed shear stress exceeds the allowable value, web reinforcement shall be provided and designed to carry the total shear force. Both vertical and horizontal shear stresses shall be considered. The area required for shear reinforcement placed perpendicular to the longitudinal reinforcement shall be computed by Eq (7.6.3) below:

$$A_v = \frac{sV}{F_s d} \quad (7.6.3)$$

Spacing of vertical shear reinforcement shall not exceed $d/2$, nor 600 mm. Inclined shear reinforcement shall have a maximum spacing of $0.375 d (1 + \cot \alpha)$, but not greater than 600 mm, where α is the acute angle between inclined bar and the horizontal.

7.6.5 Design of Members Subjected to Flexural Stress

7.6.5.1 Rectangular Elements

Rectangular flexural elements shall be designed in accordance with the following equations or other methods based on the simplified assumptions.

- (a) Compressive stress in the masonry:

$$f_b = \frac{M}{bd^2} \left(\frac{2}{jk} \right) \quad (7.6.4)$$

- (b) Tensile stress in the longitudinal reinforcement:

$$f_s = \frac{M}{A_s jd} \quad (7.6.5)$$

- (c) Design coefficients :

$$k = \left[(np)^2 + 2np \right]^{1/2} - np \quad (7.6.6)$$

or

$$k = \frac{1}{1 + \frac{f_s}{nf_b}} \quad (7.6.7)$$

$$j = 1 - \frac{k}{3} \quad (7.6.8)$$

7.6.5.2 Nonrectangular Sections

Flexural elements of nonrectangular cross-section shall be designed in accordance with the assumptions given in Sec 7.4.2.1 and 7.6.1.1.

7.6.5.3 Lateral Support

The clear distance between lateral supports of a beam shall not exceed 32 times the least depth of compression area.

7.6.5.4 Effective Width

In computing flexural stresses in walls where reinforcement occurs, the effective width assumed for running bond masonry shall not exceed 6 times the nominal wall thickness or the centre to centre distance between reinforcement. Where stack bond is used, the effective width shall not exceed 3 times the nominal wall thickness or the centre to centre distance between reinforcement or the length of one unit, unless grouted solid using open-ended joints.

7.6.5.5 Bond

In flexural members in which tensile reinforcement is parallel to the compressive face, the bond stress shall be computed by the formula:

$$u = \frac{V}{\sum_o jd} \quad (7.6.9)$$

7.6.6 Reinforcement Requirements and Details

7.6.6.1 Column Reinforcement

- (a) Vertical Reinforcement: The area of vertical reinforcement shall not be less than $0.005 A_g$ and not more than $0.04A_g$. At least four 10 mm ϕ bars shall be provided.
- (b) Lateral Ties: All longitudinal bars for columns shall be enclosed by lateral ties. Lateral support shall be provided to the longitudinal bars by the corner of a complete tie having an included angle of not more than 135 degrees or by a hook at the end of a tie. The corner bars shall have such support provided by a complete tie enclosing the longitudinal bars. Alternate longitudinal bars shall have such lateral support provided by ties and no bar shall be farther than 150 mm from such a laterally supported bar.

Lateral ties and longitudinal bars shall be placed not less than 40 mm and not more than 125 mm, from the surface of the column. Lateral ties may be against the longitudinal bars or placed in the horizontal bed joint if the requirements of Sec 4.4.6 are met. Spacing of ties shall not be more than 16 times longitudinal bar diameter, 48 times tie bar diameter or the least dimension of the column but not more than 450 mm.

Ties shall be at least 6 mm in diameter for 22 mm diameter or smaller longitudinal bars and 10 mm in diameter for larger longitudinal bars. Ties less than 10 mm in diameter may be used for longitudinal bars larger than 22 mm in diameter, provided the total cross-sectional area of such smaller ties crossing a longitudinal plane is equal to that of the larger ties at their required spacing.

- (c) Anchor Bolt Ties: Additional ties shall be provided around anchor bolts which are set in the top of the column. Such ties shall engage at least four bolts or, alternatively at least four vertical column bars or a combination of bolts and bars totaling four in number. Such ties shall be located within the top 125 mm of the column and shall provide a total of 250 square millimeters or more in cross-sectional area. The upper most ties shall be within 50 mm of the top of the column.

7.6.6.2 Maximum Reinforcement Size

The maximum size of reinforcing bars shall be 35 mm. Maximum steel area in cell shall be 6 per cent of the cell area without splices and 12 per cent of cell area with splices.

7.6.6.3 Spacing of Longitudinal Reinforcement

The clear distance between parallel bars, except in columns, shall not be less than the nominal diameter of the bars or 25 mm, except that bars in a splice may be in contact. This clear distance requirement applies to the clear distance between a contact splice and adjacent splices or bars. The minimum clear distance between parallel bars in columns shall be two and one-half times the bar diameter.

The clear distance between the surface of a bar and any surface of a masonry unit shall not be less than 6 mm for fine grout and 12 mm for coarse grout. Cross webs of hollow units may be used as support for horizontal reinforcement.

All reinforcing bars, except joint reinforcing, shall be completely embedded in mortar or grout and have a minimum cover, including the masonry unit, as specified below:

- (a) 20 mm when not exposed to weather
- (b) 40 mm when exposed to weather
- (c) 50 mm when exposed to soil

7.6.6.4 Anchorage of Flexural Reinforcement

- (a) The tension or compression in any bar at any section must be developed on each side of that section by the required development length. The development length of the bar may be achieved by a combination of an embedment length, anchorage or, for tension only, hooks.

The required development length for deformed bars or deformed wires shall be calculated by:

$$\ell_d = 0.29d_b f_s \text{ for bars in tension} \quad (7.6.10)$$

$$\ell_d = 0.22d_b f_s \text{ for bars in compression} \quad (7.6.11)$$

Development length for plain bars shall be 2.0 times the length calculated by Eq (7.6.10).

- (b) Except at supports, or at the free end of cantilevers, every reinforcing bar shall be extended beyond the point at which it is no longer needed to resist tensile stress for a distance equal to 12 bar diameters or the depth of the flexural member, whichever is greater. No flexural bars shall be terminated in a tensile zone unless one of the following conditions is satisfied:
 - i) The shear is not over one-half of that permitted, including allowance for shear reinforcement, if any.
 - ii) Additional shear reinforcement in excess of that required is provided each way from the cutoff a distance equal to the depth of the beam. The shear reinforcement

spacing shall not exceed $d/8r_b$, where r_b is the ratio of the area of bars cutoff to the total area of bars at the section.

- iii) The continuing bars provide double the area required for flexure at that point or double the perimeter required for reinforcing bond.
- (c) At least one third of the total reinforcement provided for negative moment at the support shall be extended beyond the extreme position of the point of inflection a distance sufficient to develop one half the allowable stress in the bar, one sixteenth of the clear span, or the depth d of the member, whichever is greater.
- (d) Tensile reinforcement of negative moment in any span of a continuous restrained or cantilever beam, or in any member of a rigid frame, shall be adequately anchored by reinforcing bond, hooks or mechanical anchors in or through the supporting member.
- (e) At least one third of the required positive moment reinforcement in simple beams or at the freely supported end of continuous beams shall extend along the same face of the beam into the support at least 150 mm. At least one fourth of the required positive moment reinforcement at the continuous end of continuous beams shall extend along the same face of the beam into the support at least 150 mm.
- (f) Compression reinforcement in flexural members shall be anchored by ties or stirrups not less than 6 mm in diameter, spaced not farther apart than 16 bar diameters or 48 tie diameters whichever is smaller. Such ties or stirrups shall be used throughout the distance where compression steel is required.
- (g) In regions of moment where the design tensile stresses in the steel are greater than 80 per cent of the allowable steel tensile stress (F_s), the lap length of splices shall be increased not less than 50 per cent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 per cent increase may be used.

7.6.6.5 Anchorage of Shear Reinforcement

- (a) Single separate bars used as shear reinforcement shall be anchored at each end by one of the following methods:
 - i) Hooking tightly around the longitudinal reinforcement through 180 degrees.
 - ii) Embedment above or below the mid-depth of the beam on the compression side a distance sufficient to develop the stress in the bar for plane or deformed bars.
 - iii) By a standard hook (see Sec 4.6.6.6) considered as developing 50 N/mm², plus embedment sufficient to develop the remainder of the stress to which the bars are subject. The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.
- (b) The ends of bars forming single U or multiple U stirrups shall be anchored by one of the methods specified above or shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least 12 diameters of the stirrup.
- (c) The loops or closed ends of single U or multiple U stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees and project beyond the end of the bend at least 12 diameters of the stirrup.

7.6.6.6 Hooks

- (a) The term "standard hook" shall mean one of the following:
 - i. A 180 degree turn plus an extension of at least 4 bar diameters but not less than 65 mm at the free end of the bar.
 - ii. 90 degree turn plus an extension of at least 12 bar diameters at the free end of the bar.

- iii. For stirrup and tie anchorage only either a 90 degree or a 135 degree turn, plus an extension of at least 6 bar diameters but not less than 65 mm at the free end of the bar.
- (b) The diameter of bend measured on the inside of the bar other than stirrups and ties, shall not be less than that set forth in Table 6.4.12.
- (c) Inside diameter of bend for 12 mm diameter or smaller stirrups and ties shall not be less than 4 bar diameters. Inside diameter of bend for 16 mm diameter or larger stirrups and ties shall not be less than that given in Table 6.4.12.
- (d) Hooks shall not be permitted in the tension portion of any beam, except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

Table 6.4.12: Minimum Diameter of Bend

	Bar Diameter	Minimum Diameter of Bend
1	6 mm ϕ through 25 mm ϕ	6 bar diameters
2	8 mm ϕ through 35 mm ϕ	8 bar diameters

- (e) Hooks shall not be assumed to carry a load which would produce a tensile stress in the bar greater than 50 N/mm².
- (f) Hooks shall not be considered effective in adding to the compressive resistance of bars.
- (g) Any mechanical device capable of developing the strength of the bar without damage to the masonry may be used in lieu of a hook. Data must be presented to show the adequacy of such devices.

7.6.6.7 Splices

The amount of lap of lapped splices shall be sufficient to transfer the allowable stress of the reinforcement as in Sec 4.6.6.4. In no case shall the length of the lapped splice be less than 30 bar diameters for compression and 40 bar diameters for tension.

Welded or mechanical connections shall develop 125 per cent of the specified yield strength of the bar in tension, except for connections of compression bars in columns that are not part of the seismic system and are not subject to flexure, where the compressive strength only need be developed.

When adjacent splices in grouted masonry are separated by 75 mm or less, the lap length shall be increased by 30 per cent or the splice may be staggered at least 24 bar diameters with no increase in lap length.

7.7 Strength Design of Slender Walls and Shear Walls

7.7.1 Design of Slender Walls

In lieu of the procedure set forth in Sec 4.6, the procedures prescribed in this section, which consider the slenderness of walls by representing effects of axial forces and deflection in calculation of moments, may be used when the vertical load stress at the location of maximum moment computed by Eq (7.7.1) does not exceed $0.04f'_m$. The value of f'_m shall not exceed 40 N/mm².

$$\frac{P_w + P_f}{A_g} \leq 0.04 f'_m \quad (7.7.1)$$

Slender masonry walls shall have a minimum nominal thickness of 150 mm.

7.7.1.1 Slender Wall Design Procedure

- (a) Maximum Reinforcement: The reinforcement ratio shall not exceed $0.5\rho_b$, where ρ_b is the balanced steel ratio.

- (b) **Moment and Deflection Calculation:** All moments and deflections of slender walls shall be calculated based on simple support conditions at top and bottom. For other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

7.7.1.2 Strength Design

- (a) **Loads:** Factored loads shall be determined in accordance with Chapter 2, Loads.
- (b) **Required Moment:** Required moment and axial force shall be determined at the mid-height of the wall and shall be used for design. The factored moment, M_u , at the mid-height of the wall shall be determined by Eq (7.7.2).

$$M_u = \frac{w_u h^2}{8} + P_u \frac{e}{2} + (P_{uw} + P_{uf}) \Delta_u \quad (7.7.2)$$

where :

- Δ_u = horizontal deflection at mid-height under factored load; P-Delta effects shall be included in deflection calculation.
- e = eccentricity of P_u
- P_u = axial load at mid-height of wall, including tributary wall weight.
- = $P_{uw} + P_{uf}$

- (c) **Design Strength:** Design strength in flexure is the nominal moment strength, M_n , multiplied by the strength reduction factor, ϕ and shall equal or exceed the factored moment, M_u .

$$M_u \leq \phi M_n \quad (7.7.3)$$

where :

- M_n = nominal moment strength
- = $A_{se} f_y (d - a/2)$
- A_{se} = effective area of steel
- = $\frac{A_s f_y + P_u}{f_y}$, and
- a = depth of stress block due to factored loads.
- = $\frac{P_u + A_s f_y}{0.85 f'_m b}$

The strength reduction factor ϕ for flexure shall be 0.80.

- (d) **Design Assumptions:** The following are the design assumptions for calculation of nominal strength.
- Nominal strength of singly reinforced masonry wall cross-sections subject to combined flexure and axial load shall be based on applicable conditions of equilibrium and compatibility of strains.
 - Strain in reinforcement and masonry walls shall be assumed directly proportional to the distance from the neutral axis.
 - Maximum usable strain at extreme masonry compression fibre shall be assumed equal to 0.003.
 - Stress in reinforcement below specified yield strength f_y shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y , stress in reinforcement shall be considered independent of strain and equal to f_y .
 - Tensile strength of masonry walls shall be neglected in flexural calculations of strength, except for deflection calculation.

vi) Relationship between masonry compressive stress and masonry strain may be assumed to be rectangular as defined by the following:

1. Masonry stress of $0.85f'_m$ 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 $a = 0.85c$ from the fibre of maximum compressive strain.
2. Distance c from fibre of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

7.7.1.3 Deflection Calculation

The mid-height deflection, Δ_s , under service lateral and vertical loads (without load factors) shall be limited to:

$$\Delta_s = 0.007h \quad (7.7.4)$$

The mid-height deflection shall be computed by:

$$\Delta_s = \frac{5M_s h^2}{48E_m I_g} \quad \text{when } M_{ser} \leq M_{cr} \quad (7.7.5)$$

$$\Delta_s = \frac{5M_{cr} h^2}{48E_m I_g} + 5 \frac{(M_{ser} - M_{cr}) h^2}{48E_m I_{cr}} \quad \text{when } M_{cr} < M_{ser} < M_n \quad (7.7.6)$$

The cracking moment strength of the wall M_{cr} shall be determined by:

$$M_{cr} = S f_r \quad (7.7.7)$$

The modulus of rupture, f_r , shall be determined from Table 6.7.13.

Table 6.7.13 : Values of the Modulus of Rupture, f_r

Type of Masonry	Fully Grouted	Partially Grouted
Solid Masonry	$0.17\sqrt{f'_m} \leq 0.65 \text{ N/mm}^2$	Not allowed
Hollow Unit Masonry	$0.33\sqrt{f'_m} \leq 1.2 \text{ N/mm}^2$	$0.21\sqrt{f'_m} \leq 0.65 \text{ N/mm}^2$

7.7.2 Design of Shear Walls

Based on ultimate strength design, the procedures described below may be used as an alternative to the procedure specified in Sec 4.6 for the design of reinforced hollow unit masonry shear walls. Provisions for quality control during construction of the shear wall are specified in Sec 7.3.4

7.7.2.1 Required Strength

The required strength to resist different combinations of loads shall be determined in accordance with Sec 2.7.5.1.

7.7.2.2 Design Strength

Shear walls shall be proportioned such that the design strength exceeds the required strength. Design strength in terms of axial force, shear force and moment provided by the shear wall shall be computed as the nominal strength multiplied by the strength reduction factor ϕ .

Strength reduction factor ϕ shall be as follows:

- (a) $\phi = 0.65$ for axial load and axial load with flexure

For members with f_y less than 410 N/mm^2 and with symmetrical reinforcement, ϕ may be increased linearly to 0.85 as ϕP_n decreases from $0.10f'_m A_e$ or $0.25P_b$ to zero.

For solid grouted walls P_b may be calculated by

$$P_b = 0.85 f'_m b a_b \quad (7.7.8)$$

where

$$a_b = 0.85 [e_{mu} / (e_{mu} + f_y / E_s)] d$$

(b) $\phi = 0.60$ for shear

The shear strength reduction factor may be 0.80 for any shear wall when its nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength for the factored load combination.

7.7.2.3 Design Assumptions for Nominal Strength

- (a) Nominal strength of shear wall cross-sections shall be based on assumptions specified in Sec 7.7.1.2(d).
- (b) The maximum usable strain e_{mu} , at the extreme masonry compression fibre shall not exceed 0.003.
- (c) f'_m shall not be less than 7 N/mm² or greater than 20 N/mm².

7.7.2.4 Axial Strength

The nominal axial strength of shear walls supporting axial loads only shall be calculated by Eq (7.7.9)

$$P_o = 0.85 f'_m (A_e - A_s) + f_y A_s \quad (7.7.9)$$

The shear wall shall be designed for the axial strength P_u , such that

$$P_u \leq \phi(0.80)P_o \quad (7.7.10)$$

7.7.2.5 Shear Strength

- (a) The nominal shear strength shall be determined by the provisions as specified in (b) or (c) below. The maximum nominal shear strength values are given in Table 6.7.14.

Table 6.7.14: Maximum Nominal Shear Strength Values

$\frac{M^*}{Vd}$	$\frac{V_n}{A_e \sqrt{f'_m}}$
≤ 0.25	72.0
≥ 1.00	48.0

* M is the maximum bending moment that occurs simultaneously with the shear load V at the section under consideration. Interpolation may be by straight line for M/Vd values between 0.25 and 1.00.

- (b) The nominal shear strength of shear walls except for shear walls specified in (c) below shall be determined by Eq (7.7.11).

$$V_n = V_m + V_s \quad (7.7.11)$$

where :

$$V_m = 0.083 C_d A_{mv} \sqrt{f'_m} \quad (7.7.12)$$

The value of C_d in Eq (7.7.12) is given as :

$$C_d = 2.4 \text{ for } \frac{M}{Vd} \leq 0.25$$

$$= 1.2 \text{ for } \frac{M}{Vd} \geq 1.0$$

and

$$V_s = A_{mv} \rho_n f_y \quad (7.7.13)$$

- (c) For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

- i) For all cross-sections within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined by Eq (7.7.14)

$$V_n = A_{mv} \rho_n f_y \quad (7.7.14)$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall but not to exceed one-half storey height.

- ii) For the other region, the nominal shear strength of the shear wall shall be determined by Eq (7.7.11).

7.7.2.6 Reinforcement

Reinforcement shall be in accordance with the following:

- (a) Minimum reinforcement shall be provided in accordance with Sec 7.8.5.1 for all seismic areas using this method of analysis.
- (b) When the shear wall failure mode is in flexure, the nominal flexural strength of the shear wall shall be at least 1.8 times the cracking moment strength of a fully grouted wall or 3.0 times the cracking moment strength of a partially grouted wall as obtained from Eq (7.7.7).
- (c) All continuous reinforcement shall be anchored or spliced in accordance with Sec 7.6.6.4 with $f_s = 0.5f_y$
- (d) Vertical reinforcement shall not be less than 50 per cent of the horizontal reinforcement.
- (e) Spacing of horizontal reinforcement within the region defined in Sec 7.7.2.5(c) shall not exceed three times the nominal wall thickness or 600 mm, whichever is smaller.

7.7.2.7 Boundary Member

Boundary members shall be as follows:

- (a) The need for boundary members at boundaries of shear wall shall be determined using the provisions set forth in (b) or (c) below.
- (b) Boundary members shall be provided when the failure mode is flexure and the maximum extreme fibre stress exceeds $0.2f'_m$. The boundary members may be discontinued where the calculated compressive stresses are less than $0.15f'_m$. Stresses may be calculated for the factored forces using a linearly elastic model and gross section properties.
- (c) When the failure mode is flexure, boundary member shall be provided to confine all vertical reinforcement whose corresponding masonry compressive stress exceeds $0.4f'_m$.
- (d) The minimum length of the boundary member shall be 3 times the thickness of the wall.
- (e) Boundary members shall be confined with minimum of 10 mm diameter bars at a maximum of 200 mm spacing or equivalent within the grouted core and within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall.

7.8 Earthquake Resistant Design

7.8.1 General

All masonry structures constructed in the Seismic Zones 2, 3 and 4 shown in Fig 2.5.1 shall be designed in accordance with the provisions of this Section.

7.8.2 Loads

Seismic forces on masonry structures shall be determined in accordance with the provisions of Sec 2.5 of this Part.

7.8.3 Materials

- (a) Well burnt clay bricks and concrete hollow blocks having a crushing strength not less than 12 N/mm² shall be used.
- (b) Mortar not leaner than M₃ shall be used for masonry constructions.

7.8.4 Provisions for Seismic Zone 2 and 3

7.8.4.1 Wall Reinforcement

Vertical reinforcement of at least 12 mm ϕ shall be provided continuously from support to support at each corner, at each side of each opening, at the ends of walls and at a maximum spacing of 1.2 m horizontally throughout the wall. Horizontal reinforcement not less than 12 mm ϕ shall be provided:

- (a) at the bottom and top of wall openings and shall extend at least 40 bar diameters, with a minimum of 600 mm, past the opening,
- (b) continuously at structurally connected roof and floor levels and at the top of walls,
- (c) at the bottom of the wall or in the top of the foundations when dowelled to the wall,
- (d) at maximum spacing of 3.0 m unless uniformly distributed joint reinforcement is provided. Reinforcement at the top and bottom of openings when continuous in the wall may be used in determining the maximum spacing specified in item (a) above.

7.8.4.2 Stack Bond

Where stack bond is used, the minimum horizontal reinforcement ratio shall be 0.0007 bt . This ratio shall be satisfied by uniformly distributed joint reinforcement or by horizontal reinforcement spaced not more than 1.2 m and fully embedded in grout or mortar.

7.8.4.3 Columns

Columns shall be reinforced as specified in Sec 7.6.6.1.

7.8.5 Provisions for Seismic Zone 4

All masonry structures built in Seismic Zone 4 shall be designed and constructed in accordance with requirements for Seismic Zone 2 and with the following additional requirements and limitations.

Reinforced hollow unit stack bond construction which is part of the seismic resisting system shall use open-end units so that all head joints are made solid, shall use bond beam units to facilitate the flow of grout and shall be grouted solid.

7.8.5.1 Wall Reinforcement

Reinforced masonry walls shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall and the area of reinforcement in either direction shall not be less than 0.0007 times the gross cross-sectional area of the wall. The spacing of reinforcement shall not exceed 1.20 m. The diameter of reinforcing bar shall not be less than 10 mm except that joint reinforcement may be considered as part of all of the requirements for minimum reinforcement. Reinforcement shall be continuous around wall corners and through intersections. Only reinforcement which is continuous in the wall or element shall be considered in computing the minimum area of reinforcement. Reinforcement with splices conforming to Sec 7.6.6.7 shall be considered as continuous reinforcement.

7.8.5.2 Column Reinforcement

The spacing of column ties shall be not more than 225 mm for the full height of columns stressed by tensile or compressive axial overturning forces due to the seismic loads, and 225 mm for the tops and

bottoms of all other columns for a distance of one sixth of the clear column height, but not less than 450 mm or maximum column dimension. Tie spacing for the remaining column height shall be not more than 16 bar diameters, 48 tie diameters or the least column dimension, but not more than 450 mm.

7.8.5.3 Stack Bond

Where stack bond is used, the minimum horizontal reinforcement ratio shall be $0.0015bt$. If open-end units are used and grouted solid, the minimum horizontal reinforcement ratio shall be $0.0007bt$.

7.8.5.4 Minimum Dimension

- (a) Bearing Walls: The nominal thickness of reinforced masonry bearing walls shall be not less than 150 mm except that nominal 100 mm thick load bearing reinforced hollow clay unit masonry walls may be used, provided net area unit strength exceeds 55 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 flush cut, concave or a protruding V section.
- (b) Columns: The least nominal dimension of a reinforced masonry column shall be 375 mm except that if the allowable stresses are reduced to 50 per cent of the values given in Sec 4.3, the minimum nominal dimension shall be 250 mm.

7.8.5.5 Shear Wall

- (a) When calculating shear or diagonal tension stresses, shear walls which resist seismic forces shall be designed to resist 1.5 times the forces specified in Chapter 2, Loads.
- (b) The portion of the reinforcement required to resist shear shall be uniformly distributed and shall be joint reinforcing, deformed bars, or a combination thereof. The maximum spacing of reinforcement in each direction shall be not less than the smaller of one-half the length or height of the element or more than 1.20 m.

Joint reinforcement used in exterior walls and considered in the determination of the shear strength of the member shall conform to the requirement "Joint Reinforcement for Masonry" (UBC Standard No. 24-15) or "Standard Specification for Steel Wire, Plain, for Concrete Reinforcement", (ASTM, A82).

Reinforcement required to resist in-plane shear shall be terminated with a standard hook or with an extension of proper embedment length beyond the reinforcing at the end of the wall section. The hook or extension may be turned up, down or horizontally. Provisions shall be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams shall be fully anchored into these elements.

- (c) Multi-wythe grouted masonry shear walls shall be designed with consideration of the adhesion bond strength between the grout and masonry units. When bond strengths are not known from previous tests, the bond strength shall be determined by test.

7.8.5.6 Hook

The standard hook for tie anchorage shall have a minimum turn of 135 degrees plus an extension of at least 6 bar diameters, but not less than 100 mm at the free end of the bar. Where the ties are placed in the horizontal bed joints, the hook shall consist of a 90 degree bend having a radius of not less than 4 tie diameters plus an extension of 32 tie diameters.

7.8.5.7 Mortar Joints Between Masonry and Concrete

Concrete abutting structural masonry such as at starter courses or at wall intersections not designed as true separation joints shall be roughened to a full amplitude of 1.5 mm and shall be bonded to the masonry as per the requirements of this section as if it were masonry.

7.8.6 Additional Requirements

7.8.6.1 Opening in Bearing Walls

- (a) Tops of all openings in a storey shall preferably be at the same level so that a continuous band could be provided over them, including the lintels throughout the building.
- (b) The total width of the openings shall not be more than half of the length of the walls between the adjacent cross walls, except as provided in (f) below.
- (c) The opening shall preferably be located away from the corner by a clear distance equal to at least one-eighth of the height of the opening for Seismic Zone 2 and 3 and one-fourth of the height for Seismic Zone 4.
- (d) The horizontal distance between two openings shall not be less than one-fourth of the height of the shorter opening for Seismic Zone 2 and 3 and one-half of the height for Seismic Zone 4.
- (e) The vertical distance between openings one above the other shall be not less than 600 mm.
- (f) Where openings do not comply with the requirements of (b) and (c) above, they shall be strengthened in accordance with Sec 7.8.6.5.
- (g) If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored.
- (h) If an opening is tall say, for the full height of wall, dividing the wall into two portions, these portions shall be reinforced with horizontal reinforcement of 6 mm diameter bars at not more than 600 mm intervals, one on inner and one on outer face, properly tied to vertical steel at jambs and corners or junctions of walls where used.
- (i) The use of arches to span over the openings is a source of weakness and shall be avoided unless steel ties are provided.

7.8.6.2 Strengthening Arrangements

All masonry buildings shall be strengthened by the methods specified in Table 6.7.15.

Table 6.7.15 : Strengthening of Masonry Buildings for Earthquake

Seismic Zones	No. of Storey	Strengthening Arrangements to be Provided.
1	Up to 4	a) Masonry mortar shall not be leaner than M3
2	Up to 2 with pitched roof	a) Masonry mortar shall not be leaner than M3 b) By lintel and roof band (Sec 7.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 7.8.6.4) d) Bracing in plan at tie level for pitched roof*
	3 to 4	a) Masonry mortar shall not be leaner than M3 b) By lintel and roof band (Sec 7.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 7.8.6.4) d) Vertical reinforcement at jambs of openings (Sec 7.8.6.5) e) Bracing in plan at tie level for pitched roof*

3	Up to 4	a) Masonry mortar shall not be leaner than M3 b) By lintel and roof band (Sec 7.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 7.8.6.4) d) Vertical reinforcement at jambs of openings (Sec 7.8.6.5) e) Bracing in plan at tie level for pitched roof*
At tie level all the trusses and the gable end shall be provided with diagonal bracing in plan so as to transmit the lateral shear due to earthquake force to the gable walls acting as shear walls at the ends.		

7.8.6.3 Bands

Roof band need not be provided underneath reinforced concrete or brickwork slabs resting on bearing walls, provided the slabs are continuous over parts between crumple sections, if any, and cover the width of end walls fully.

The band shall be made of reinforced concrete with f'_c not less than 20 N/mm² or reinforced brickwork in cement mortar not leaner than 1: 4. The bands shall be to the full width of the wall and not less than 75 mm in depth and shall be reinforced as indicated in Table 6.7.16. In case of reinforced brickwork, the thickness of joints containing steel bars shall be increased so as to have a minimum mortar cover of 6 mm around the bar. In bands of reinforced brickwork, the area of steel provided shall be equal to that specified above for reinforced concrete bands.

Table 6.7.16 : Band Reinforcement

Seismic Zones	Reinforcement		
	Plain Mild Steel Bars	High Strength Deformed Bars	Links
2	2 - 12 mm ϕ , one on each face of the wall with suitable cover	2 - 10 mm ϕ , one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c
3	2 - 16 mm ϕ , one on each face of the wall with suitable cover	2 - 12 mm ϕ , one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c

7.8.6.4 Strengthening of Corner and Junctions

Vertical steel at corners and junctions of walls which are up to one and a half bricks thick shall be provided either with mild steel or high strength deformed bars as specified in Table 6.7.17. For thicker walls, reinforcement shall be increased proportionately. The reinforcement shall be properly embedded in the plinth masonry of foundations and roof slab or roof band so as to develop its tensile strength in bond and passing through the lintel bands in all storeys. Bars in different storeys may be welded or suitably lapped.

- Typical details of vertical steel in brickwork and hollow block at corners, T-junctions and jambs of opening are shown in Fig 6.7.1 and Fig 6.7.2.
- Details of vertical reinforcement given in Table 6.7.17 are applicable to brick masonry and hollow block masonry.

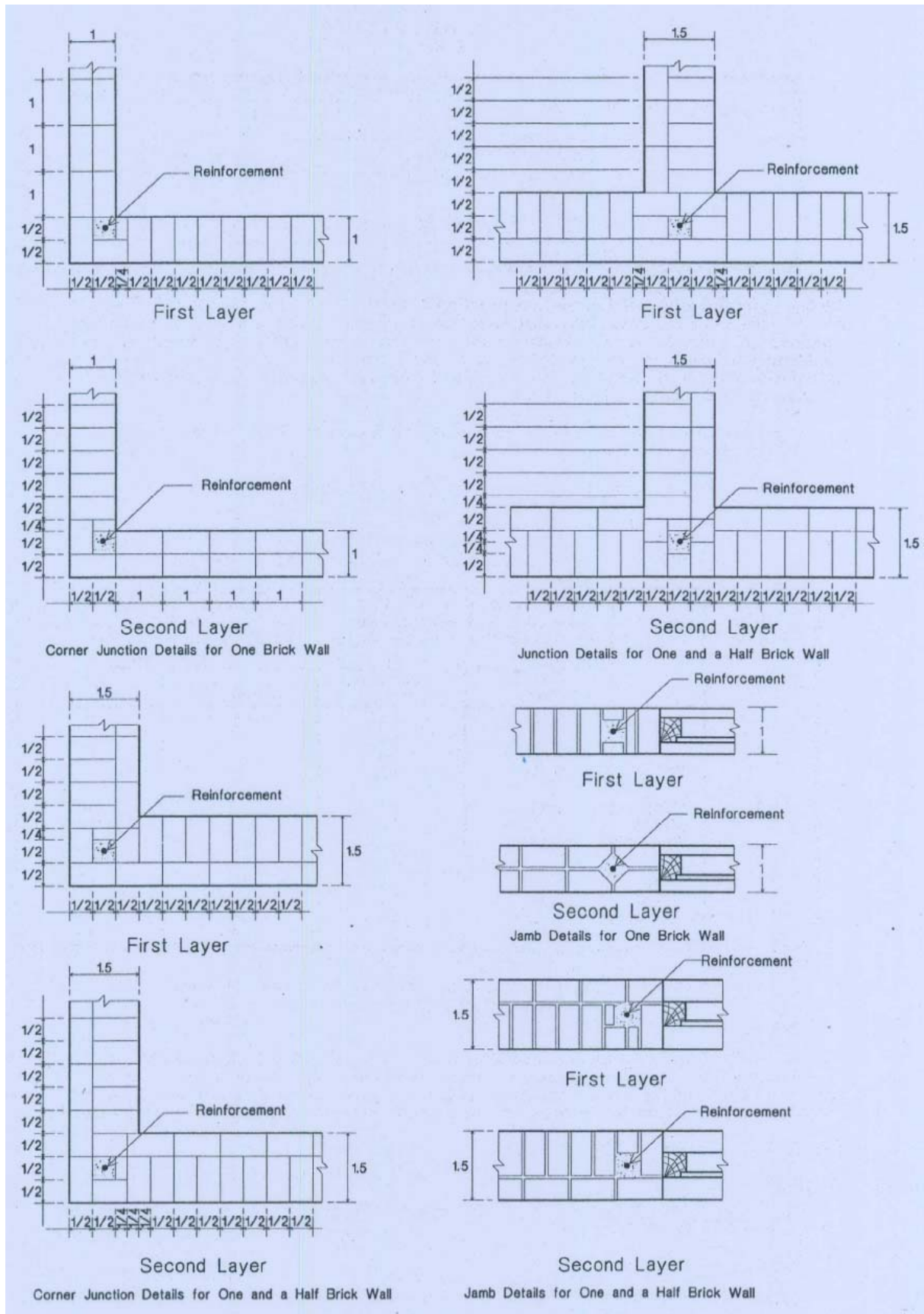


Fig. 6.7.1: Typical details of vertical reinforcement in brick masonry

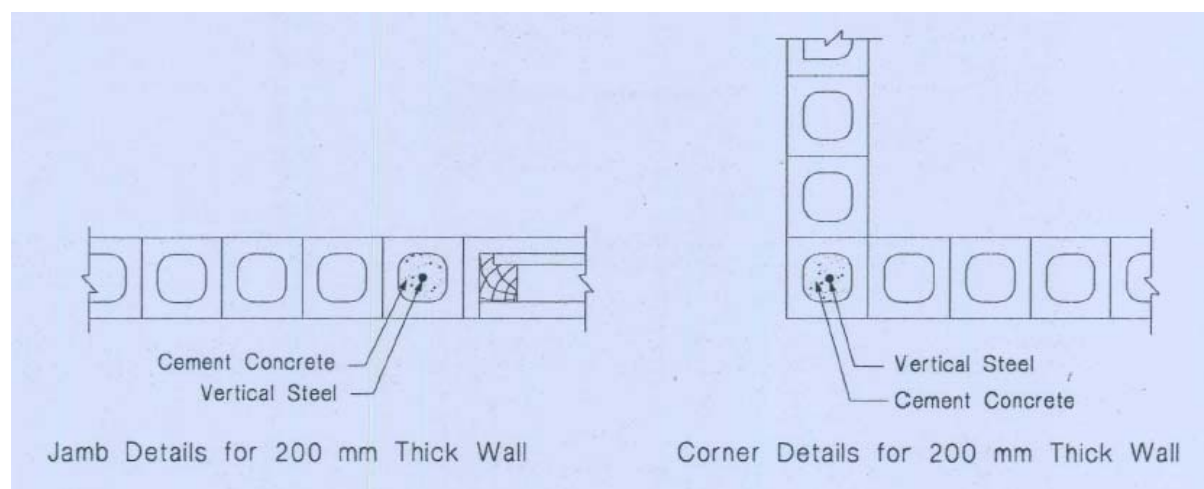


Fig. 6.7.2: Typical details of vertical reinforcement in hollow block masonry

Table 6.7.17: Vertical Reinforcement for Brick and Hollow Block Masonry

No. of Storeys	Storeys	Diameter of Single Bar or Equivalent Area of Plain Mild Steel Bar to be Provided		Diameter of Single Bar or Equivalent Area of High Strength Deformed Bar to be Provided	
		Zone 2 & 3	Zone 4	Zone 2 & 3	Zone 4
		(mm)	(mm)	(mm)	(mm)
1	-	nil	12	nil	10
2	Top	nil	12	nil	10
	Bottom	nil	16	nil	12
3	Top	12	12	10	10
	Middle	12	16	10	12
	Bottom	16	16	12	12
4	Top	12	12	10	10
	Third	12	16	10	12
	Second	16	20	12	16
	Bottom	16	25	12	20

7.8.6.5 Strengthening of Jambs of Openings

Openings in bearing walls shall be strengthened, where necessary, by providing reinforced concrete members or reinforcing the brickwork around them as shown in Fig 6.7.3.

7.8.6.6 Walls Adjoining Structural Framing

Where walls are dependent on the structural frame for lateral support they shall be anchored to the structural members with metal ties or keyed to the structural members. Horizontal ties shall consist of 6 mm diameter U-bars spaced at a maximum of 450 mm on centre and embedded at least 250 mm into the masonry and properly tied to the vertical steel of the same member.

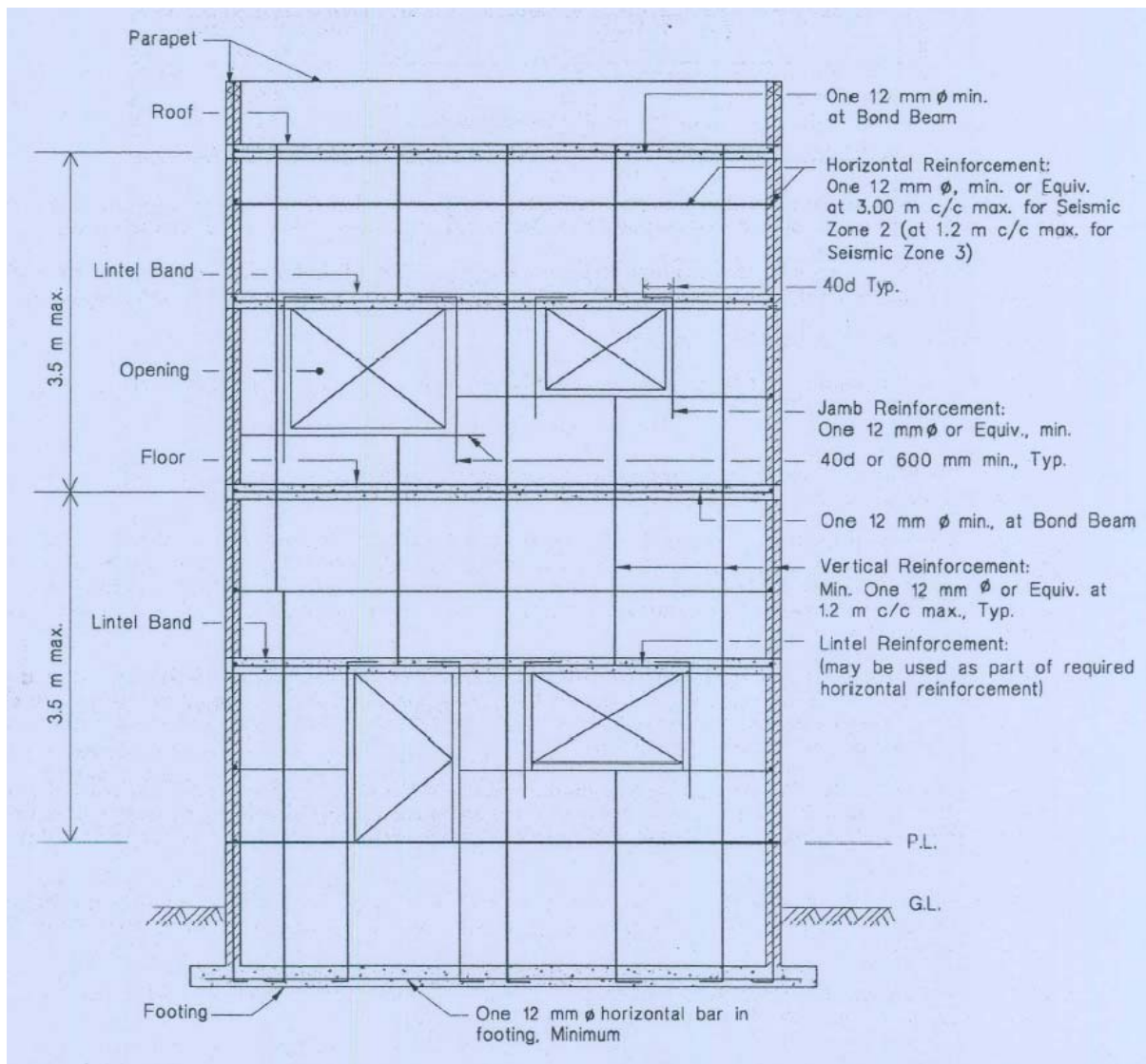


Fig. 6.7.3: Minimum reinforcement in walls and around openings in Seismic Zones 2, 3 and 4

7.9 PROVISIONS FOR HIGH WIND REGIONS

7.9.1 General

The provisions of this section shall apply to masonry structures located at regions where the basic wind speed is greater than 200 km/h.

7.9.2 Materials

Materials for masonry structures shall generally comply with the provisions of Part 5; however, there are some special requirements for masonry construction in high wind regions, which are given below:

- Burnt clay bricks shall have a compressive strength not less than 15 N/mm^2 ,
- Grout shall have a minimum compressive strength of 12.5 N/mm^2 ,
- Mortar for exterior walls and interior shear walls shall be type M_1 or M_2 ,
- Un burnt clay masonry units shall not be used.

7.9.3 Construction Requirements

Masonry construction shall comply with the provisions of Sec 7.10.

7.9.4 Foundation

Footings shall have a thickness of not less than 375 mm and shall be extended 450 mm below the undisturbed ground surface. Foundation stem wall shall have the same width and reinforcement as the wall it supports.

7.9.5 Drainage

Walls retaining more than 1 m of earth and enclosing interior spaces or floors below grade shall have minimum 100 mm diameter footing drain. A slope of 1:50 away from the building shall be provided around the building.

7.9.6 Wall Construction

7.9.6.1 Minimum thickness of different types of wall shall be as given in Table 6.7.18.

Table 6.7.18: Minimum thickness of Walls in High Wind Region

Type of Wall	Minimum Thickness (mm)
Unreinforced grouted brick wall	250
Reinforced exterior bearing wall	200
Unreinforced hollow and solid masonry wall	200
Interior nonbearing wall	150

7.9.6.2 All walls shall be laterally supported at the top and bottom. The maximum unsupported height of bearing walls or other masonry walls shall be 3.5 m. Gable end walls may be 4.5 m high at their peak.

7.9.6.3 The span of lintels over openings shall not exceed 3.5 m. All lintels shall be reinforced and the reinforcement bars shall extend not less than 600 mm beyond the edge of opening and into lintel supports.

7.9.6.4 Walls shall be adequately reinforced.

7.9.6.5 Anchors between walls and floors or roofs shall be embedded in grouted cells or cavities and shall conform to Sec 7.9.7 below.

7.9.7 Floor and Roof Systems

Floors and roofs of all masonry structures shall be adequately anchored with the wall it supports to resist lateral and uplift forces due to wind specified in Sec 2.4 of this Part.

7.9.8 Lateral Force Resistance

7.9.8.1 Strapping, approved framing anchors and mechanical fasteners, bond beams and vertical reinforcement shall be installed to provide a continuous tie from the roof to foundation system as shown in Fig 6.7.4. In addition, roof and floor systems, masonry shear walls, or masonry or wood cross walls shall be provided for lateral stability.

7.9.8.2 Floor and roof diaphragms shall be properly connected to masonry walls. Gable and sloped roof members not supported at the ridge shall be tied by the ceiling joist or equivalent lateral ties located as close to where the roof members bear on the wall as practically possible and not at more than 1.2 m on centers. Collar ties shall not be used for these lateral ties.

7.9.8.3 Masonry walls shall be provided around all sides of floor and roof systems in accordance with Fig 6.7.5. The cumulative length of exterior masonry walls along each side of the floor or roof systems

shall be at least 20 per cent of the parallel dimension. Required elements shall be without openings and shall not be less than 1.25 m in width.

Interior cross walls at right angles to bearing walls shall be provided when the length of the building perpendicular to the span of the floor or roof framing exceeds twice the distance between shear walls or 10 m, whichever is greater.

- 7.9.8.4 When required interior cross wall shall be at least 1.8 m long and reinforced with 2 mm wire joint reinforcement spaced not more than 400 mm on centre.

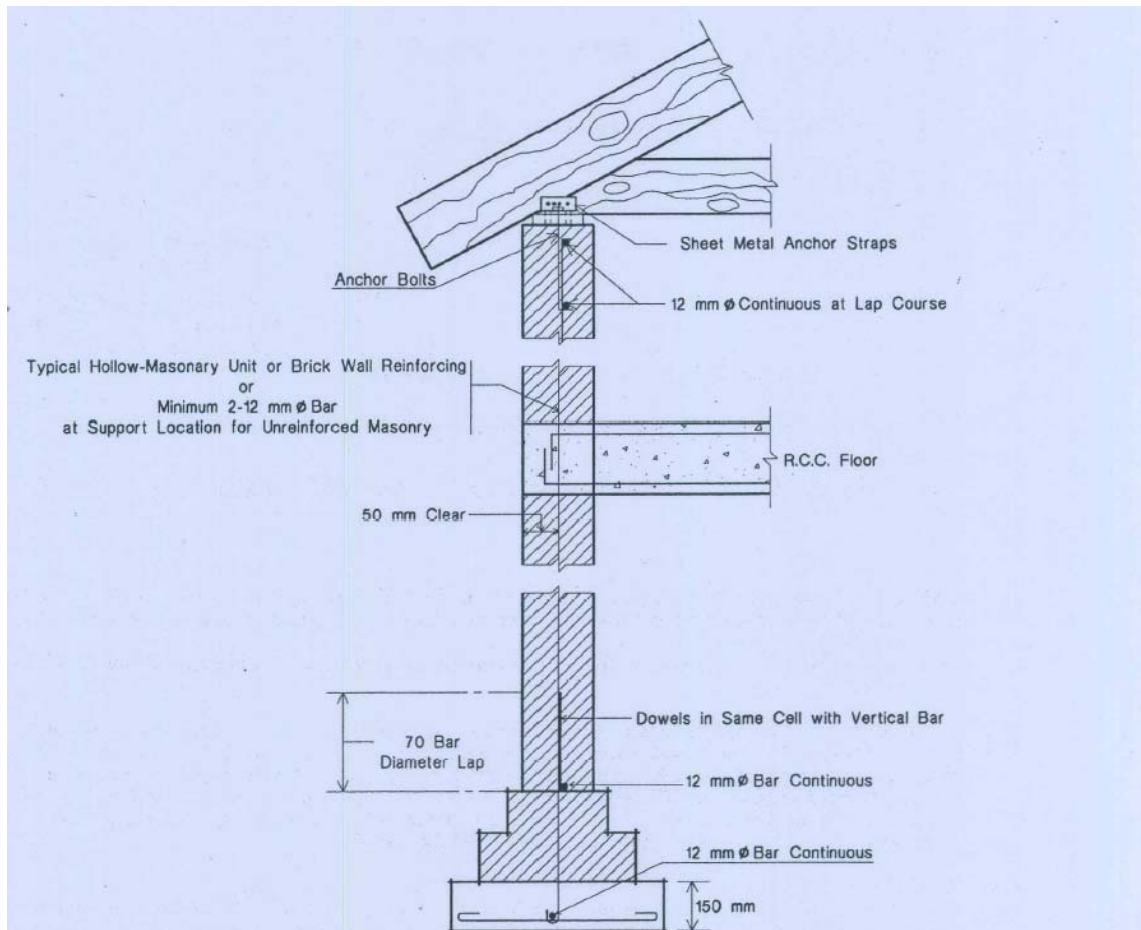


Fig. 6.7.4: Continuous tie from roof to foundation of masonry structure

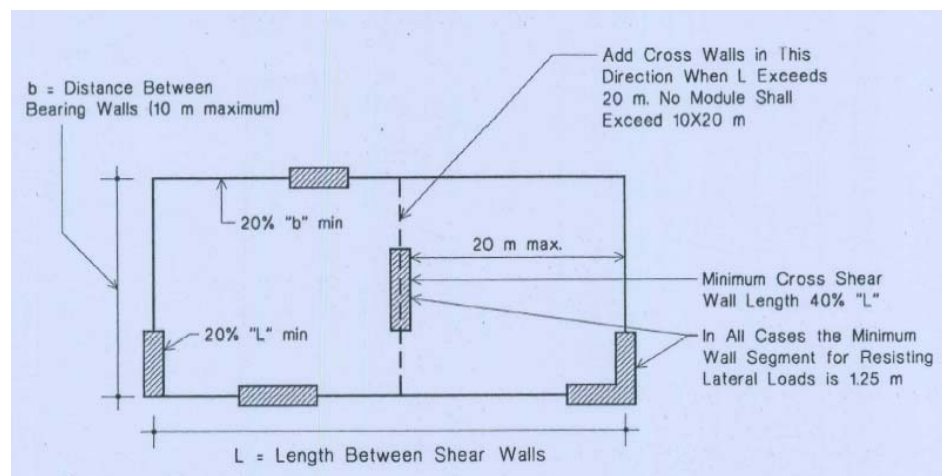


Fig. 6.7.5: Masonry walls required in high wind regions

7.10 CONSTRUCTION

7.10.1 General

Masonry shall be constructed according to the provisions of this section.

7.10.2 Storage and Preparation of Construction Materials

Storage, handling and preparation at the site shall conform to the following:

- (a) Masonry materials shall be stored in such a way that at the time of use the materials are clean and structurally suitable for the intended use.
- (b) All metal reinforcement shall be free from loose rust and other coatings that would inhibit reinforcing bond.
- (c) Burnt clay units shall have a rate of absorption per minute not exceeding one litre per square metre at the time of lying. In the absorption test the surface of the unit shall be held 3 mm below the surface of the water.
- (d) Burnt clay units shall be thoroughly wetted before placing. Concrete masonry units shall not be wetted unless otherwise approved.
- (e) Materials shall be stored in such a manner that deterioration or intrusion of foreign materials is prevented and at the time of mixing the material conforms to the applicable requirements.
- (f) The method of measuring materials for mortar and grout shall be such that proportions of the materials can be easily controlled.
- (g) Mortar or grout mixed at the job site shall be mixed for a period of time not less than 3 minutes or more than 10 minutes in a mechanical mixer with the amount of water required to provide the desired workability. Hand mixing of small amounts of mortar is permitted. Mortar may be retempered. Mortar or grout which has hardened or stiffened due to hydration of the cement shall not be used, but under no case shall mortar be used two and one-half hours, nor grout used one and one-half hours, after the initial mixing water has been added to the dry ingredients at the job site.

7.10.3 Placing Masonry Units

- (a) The mortar shall be sufficiently plastic and units shall be placed with sufficient pressure to extrude mortar from the joint and produce a tight joint. Deep furrowing which produces voids shall not be used.
The initial bed joint thickness shall not be less than 5 mm or more than 25 mm; subsequent bed joints shall be not less than 5 mm or more than 15 mm in thickness.
- (b) All surfaces in contact with mortar or grout shall be clean and free of deleterious materials.
- (c) Solid masonry units shall have full head and bed joints.
- (d) All head and bed joints shall be filled solidly with mortar for a distance from the face of the unit not less than the thickness of the shell.
Head joints of open-end units with beveled ends need not be mortared. The beveled ends shall form a grout key which permits grout within 16 mm of the face of the unit. The units shall be tightly butted to prevent leakage of grout.

7.10.4 Verticality and Alignment

All masonry shall be built true and plumb within the tolerances prescribed below. Care shall be taken to keep the perpend properly aligned.

- (a) Deviation from vertical within a storey shall not exceed 6 mm per 3m height.
- (b) Deviation in verticality in total height of any wall of a building more than one storey in height shall not exceed 12 mm.
- (c) Deviation from position shown on plan of any brickwork shall not exceed 12 mm.
- (d) Relative displacement between load bearing walls in adjacent storeys intended to be in vertical alignment shall not exceed 6 mm.

- (e) Deviation of bed joint from horizontal in a length of 12 m shall not exceed 6 mm subject to a maximum deviation of 12 mm.
- (f) Deviation from the specified thickness of bed joints, cross joints and perpend shall not exceed one-fifth of the specified thickness.

7.10.5 Reinforcement Placing

Reinforcing details shall conform to the requirements of Sec 4.6.6. Metal reinforcement shall be located in accordance with the plans and specifications. Reinforcement shall be secured against displacement prior to grouting by wire positioners or other suitable devices at intervals not exceeding 20 bar diameters.

Tolerances for the placement of steel in walls and flexural elements shall be ± 12 mm for $d \leq 200$ mm, ± 25 mm for $200 \text{ mm} \leq d \leq 600$ mm and ± 30 mm for $d > 600$ mm. Tolerance for longitudinal location of reinforcement shall be ± 50 mm.

7.10.6 Grouted Masonry

Grouted masonry shall be constructed in such a manner that all elements act together as a structural element.

Space to be filled with grout shall be clean and shall not contain any foreign materials. Grout materials and water content shall be controlled to provide adequate workability and shall be mixed thoroughly. The grouting of any section of wall shall be completed in one day with no interruptions greater than one hour.

Size and height limitations of the grout space or cell shall not be less than those shown in Table 6.7.19. Higher grout pours or smaller cavity widths or cell size than shown in Table 6.7.19 may be used when approved, if it can be demonstrated that grout spaces are properly filled.

Cleanouts are required for all grout pours over 1.5 m in height. When required, cleanouts shall be provided in the bottom course at every vertical bar but shall not be spaced more than 800 mm on centre for solidly grouted masonry. When cleanouts are required, they shall be sealed after inspection and before grouting. When cleanouts are not provided, special provisions must be made to keep the bottom and sides of the grout spaces, as well as the minimum total clear area as required by Table 6.7.19, clean and clear prior to grouting.

Table 6.7.19: Grouting Limitations

Grout Type	Grout pour Maximum Height (m)	Minimum Dimensions of the Total Clear Areas within Grout Spaces and Cells	
		Multi-wythe Masonry (mm)	Hollow Unit Masonry (mm)
Fine	0.30	20	40 × 50
Fine	1.50	40	40 × 50
Fine	2.40	40	40 × 75
Fine	3.65	40	45 × 75
Fine	7.30	50	75 × 75
Coarse	0.30	40	40 × 75
Coarse	1.50	50	60 × 75
Coarse	2.40	50	75 × 75
Coarse	3.65	60	75 × 75
Coarse	7.30	75	75 × 100

7.10.7 Chases, Recesses and Holes

- (a) Chases, recesses and holes may be permitted in masonry provided either they are considered in the structural design or they are not cut into walls made of hollow or perforated units, or vertical chases are planned instead of horizontal chases.

- (b) Depth of vertical and horizontal chases in load bearing walls shall not exceed one-third and one-sixth of the wall thickness respectively.
- (c) Vertical chases shall not be closer than 2 m in any stretch of wall and shall not be located within 350 mm of an opening or within 230 mm of a cross wall that serves as stiffening wall for stability. Width of a vertical chase shall not exceed the thickness of wall in which it occurs.
- (d) Horizontal chases shall be located in the upper or lower middle third height of wall at a distance not less than 600 mm from lateral support. No horizontal chase shall exceed one metre in length and there shall not be more than 2 chases in any one wall. Horizontal chases shall have minimum mutual separation distance of 500 mm. Sum of lengths of all chases and recesses in any horizontal plane shall not exceed one-fourth the length of the wall.
- (e) Lintel shall not be used to support masonry directly above a recess or a hole wider than 300 mm. No lintel however, is necessary in case of a circular recess or hole exceeding 300 mm in diameter provided upper half of the recess or hole is built as a semi-circular arch of adequate thickness and there is adequate length of masonry on the sides of openings to resist the horizontal thrust.
- (f) Recesses and holes in masonry shall be kept at the time of construction so as to avoid subsequent cutting. If cutting is necessary, it shall be done using sharp tools without causing heavy impact and damage to the surrounding areas.
- (g) No chase, recess or hole shall be provided in half-brick load bearing wall, excepting the minimum number of holes needed for scaffolding.

7.11 CONFINED MASONRY

7.11.1 General

Confined masonry construction consists of masonry walls (made either of clay brick or concrete block units) and horizontal and vertical RC confining members built on all four sides of a masonry wall panel. Vertical members, called tie-columns or practical columns, resemble columns in RC frame construction except that they tend to be of far smaller cross-section. Horizontal elements, called tie-beams, resemble beams in RC frame construction. To emphasize that confining elements are not beams and columns, alternative terms horizontal ties and vertical ties could be used instead of tie-beams and tie-columns.

The confining members are effective in

- (a) Enhancing the stability and integrity of masonry walls for in-plane and out-of-plane earthquake loads (confining members can effectively contain damaged masonry walls),
- (b) Enhancing the strength (resistance) of masonry walls under lateral earthquake loads, and
- (c) Reducing the brittleness of masonry walls under earthquake loads and hence improving their earthquake performance.

The structural components of a confined masonry building are (see Figure 6.7.6):

- (a) Masonry walls – transmit the gravity load from the slab(s) above down to the foundation. The walls act as bracing panels, which resist horizontal earthquake forces. The walls must be confined by concrete tie-beams and tie-columns to ensure satisfactory earthquake performance.
- (b) Confining elements (tie-columns and tie-beams) – provide restraint to masonry walls and protect them from complete disintegration even in major earthquakes.

These elements resist gravity loads and have important role in ensuring vertical stability of a building in an earthquake.

- (a) Floor and roof slabs – transmit both gravity and lateral loads to the walls. In an earthquake, slabs behave like horizontal beams and are called diaphragms.
- (b) Plinth band – transmits the load from the walls down to the foundation. It also protects the ground floor walls from excessive settlement in soft soil conditions.
- (c) Foundation – transmits the loads from the structure to the ground.

The design of confined masonry members shall be based on similar assumptions to those set out for unreinforced and for reinforced masonry members. Confined masonry shall be constructed according to the provisions of this section.

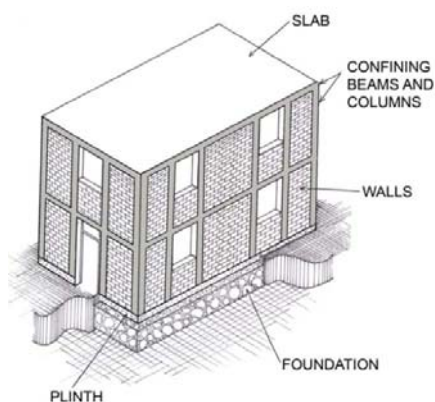


Fig. 6.7.6: A typical confined masonry building

7.11.2 Difference of Confined Masonry from RC Frame Construction

The appearance of a finished confined masonry construction and a RC frame construction with masonry in fills may look alike, however these two construction systems are substantially different. The main differences are related to the construction sequence, as well as to the manner in which these structures resist gravity and lateral loads. These differences are summarized in Table 6.7.20 and are illustrated by diagrams in Figure 6.7.7.

In confined masonry construction, confining elements are not designed to act as a moment-resisting frame; as a result, detailing of reinforcement is simple. In general, confining elements have smaller cross-sectional dimensions than the corresponding beams and columns in a RC frame building. It should be noted that the most important difference between the confined masonry walls and infill walls is that infill walls are not load-bearing walls, while the walls in a confined masonry building are.

A transition from RC frame to confined masonry construction in most cases leads to savings related to concrete cost, since confining elements are smaller in size than the corresponding RC frame members.

Table 6.7.20. A comparison between the confined masonry and RC frame construction

	Confined masonry	RC frame construction
Gravity and lateral load-resisting system	Masonry walls are the main load bearing elements and are expected to resist both gravity and lateral loads. Confining elements (tie-beams and tie-columns) are significantly smaller in size than RC beams	RC frames resist both gravity and lateral loads through their relatively large beams, columns, and their connections. Masonry in fills are <u>not</u> load-bearing walls
Foundation construction	Strip footing beneath the wall and the RC plinth band	Isolated footing beneath each column

	Confined masonry	RC frame construction
Superstructure construction sequence	1. Masonry walls are constructed first. 2. Subsequently, tie-columns are cast in place. 3. Finally, tie-beams are constructed on top of the walls, simultaneously with the floor/roof slab construction.	1. The frame is constructed first. 2. Masonry walls are constructed at a later stage and are not bonded to the frame members; these walls are nonstructural, that is, non-

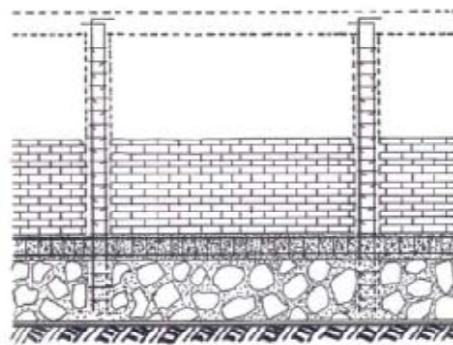
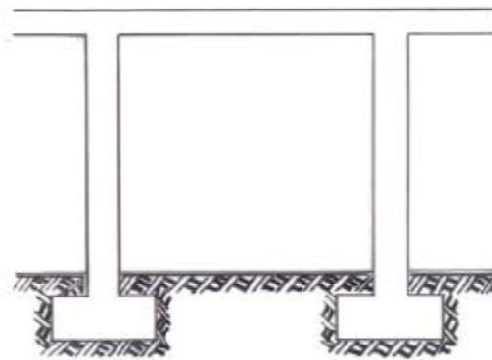


Fig. 6.7.7: RC frame construction (top) and confined masonry construction (bottom)

7.11.3 Mechanism of Resisting Earthquake Effects

A confined masonry building subjected to earthquake ground shaking can be modeled as a vertical truss, as shown in Figure 6.7.8 (left). Masonry walls act as diagonal struts subjected to compression, while reinforced concrete confining members act in tension and/or compression, depending on the direction of lateral earthquake forces. This model is appropriate before the cracking in the walls takes place. Subsequently, the cracking is concentrated at the ground floor level and significant lateral deformations take place. Under severe earthquake ground shaking, the collapse of confined masonry buildings may take place due to soft storey effect similar to the one observed in RC frames with masonry in fills, as shown in Figure 6.7.8 (right).

The following failure modes are characteristic of confined masonry walls:

- (a) Shear failure mode, and
- (b) Flexural failure mode.

Note that, in confined masonry structures, shear failure mode develops due to in-plane seismic loads (acting along in the plane of the wall), whereas flexural failure mode may develop either due to in-plane or out-of-plane loads (acting perpendicular to the wall plane).

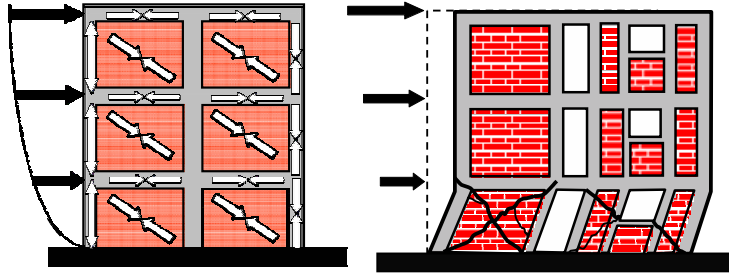


Fig. 6.7.8: Confined masonry building: vertical truss model (left) and collapse at the ground floor level (right)

Shear failure mode is characterized by distributed diagonal cracking in the wall. These cracks propagate into the tie-columns at higher load levels, as shown in Figure 6.7.9. Initially, a masonry wall panel resists the effects of lateral earthquake loads by itself while the confining elements (tie-columns) do not play a significant role. However, once the cracking takes place, the wall pushes the tie-columns sideways. At that stage, vertical reinforcement in tie-columns becomes engaged in resisting tension and compression stresses. Damage in the tie-columns at the ultimate load level is concentrated at the top and the bottom of the panel. These locations, characterized by extensive crushing of concrete and yielding of steel reinforcement, are called plastic hinges (Figure 6.7.10). Note that the term plastic hinge has a different meaning in the context of confined masonry components than that referred to in relation to RC beams and columns, where these hinges form due to flexure and axial loads. In confined masonry construction, tie-beams and tie-columns resist axial loads. Shear failure can lead to severe damage in the masonry wall and the top and bottom of the tie-columns.

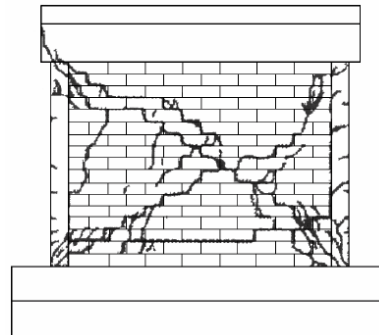


Fig. 6.7.9: Shear failure of confined masonry walls

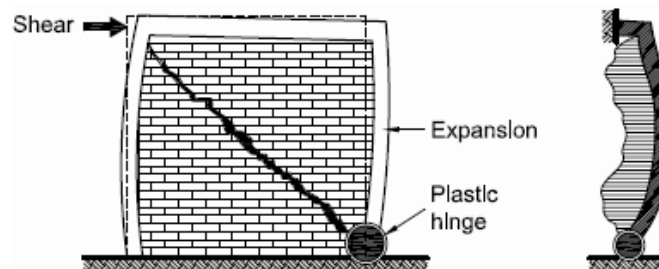


Fig. 6.7.10: Plastic hinge developed in a confined masonry wall

Flexural failure caused by in-plane lateral loads is characterized by horizontal cracking in the mortar bed joints on the tension side of the wall, as shown in Figure 6.7.11. Extensive horizontal cracking, which usually takes place in tie-columns, as well as shear cracking can be observed.

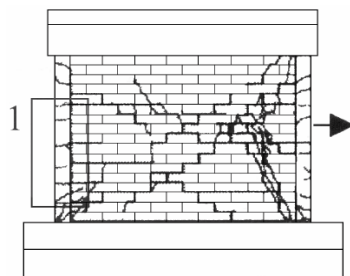


Fig. 6.7.11: Flexural failure of confined masonry walls

Irrespective of the failure mechanism, tie-columns resist the major portion of gravity load when masonry walls suffer severe damage (this is due to their high axial stiffness and load resistance). The failure of a tie-column usually takes place when cracks propagate from the masonry wall into the tie-column and shear it off. Subsequently, the vertical stability of the entire wall is compromised. Vertical strains in the confined masonry walls decrease at an increased damage level, thereby indicating that a major portion of the gravity load is resisted by tie-columns. This finding confirms the notion that tie-columns have a critical role in resisting the gravity load in damaged confined masonry buildings and ensuring their vertical stability.

7.11.4 Key Factors Influencing Seismic Resistance

7.11.4.1 Wall Density

Wall density is believed to be one of the key parameters influencing the seismic performance of confined masonry buildings. It can be determined as the transverse area of walls in each principal direction divided by the total floor area of the building.

7.11.4.2 Masonry Units and Mortar

The lateral load resistance of confined masonry walls strongly depends on the strength of the masonry units and the mortar used. The walls built using low-strength bricks or ungrouted hollow block units had the lowest strength while the ones built using grouted or solid units had the largest strength. However, the use of grouted and solid units results in an increase both in wall mass and seismic loads. Also, the weaker the mortar the lower the masonry strength (due to the unit-mortar interaction, the masonry strength is always lower than the unit strength). There is no significant difference in strength between unreinforced and confined masonry wall specimens with the same geometry and material properties.

7.11.4.3 Tie-Columns

Tie-columns significantly influence the ductility and stability of cracked confined masonry walls. The provision of closely spaced transverse reinforcement (ties) at the top and bottom ends of tie-columns results in improved wall stability and ductility in the post-cracking stage.

7.11.4.4 Horizontal Wall Reinforcement

Horizontal reinforcement has a beneficial effect on wall ductility. Specimens with horizontal reinforcement showed a more uniform distribution of inclined shear cracks than the unreinforced specimens. Horizontal re bars should be anchored into the tie-columns; the anchorage should be provided with 90° hooks at the far end of the tie-column (see Figure 6.7.12). The hooks should be embedded in the concrete within the tie-column (note that the tie-column reinforcement was omitted from the figure). The bar diameter should be larger than 3.5 mm and less than $\frac{1}{4}$ the joint thickness.

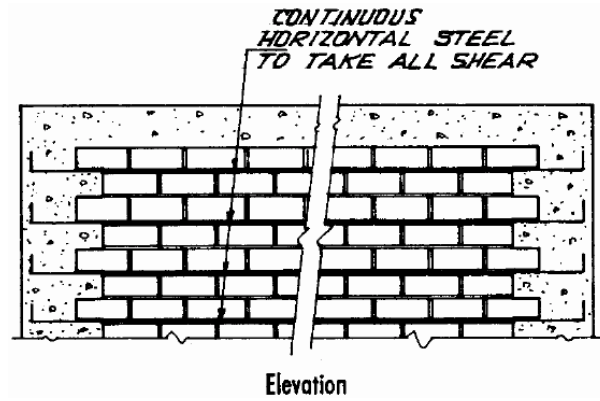


Fig. 6.7.12: Horizontal reinforcement in confined masonry walls

7.11.4.5 Openings

When the opening area is less than approximately 10% of the total wall area, the wall lateral load resistance is not significantly reduced as compared to a solid wall (i.e. wall without openings). The walls with larger openings develop diagonal cracks (same as solid walls), except that the cracks are formed in the piers between the openings; thus, diagonal struts form in the piers, as shown in Figure 6.7.13.

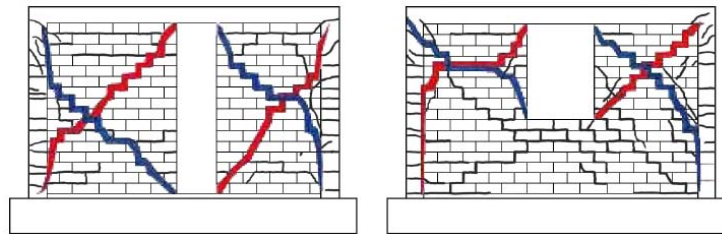


Fig. 6.7.13: Failure modes in the confined masonry walls with openings

7.11.5 Verification of Members

7.11.5.1 In the verification of confined masonry members subjected to bending and/or axial loading, the assumptions for reinforced masonry members should be adopted. In determining the design value of the moment of resistance of a section a rectangular stress distribution may be assumed, based on the strength of the masonry, only. Reinforcement in compression should also be ignored.

7.11.5.2 In the verification of confined masonry members subjected to shear loading the shear resistance of the member should be taken as the sum of the shear resistance of the masonry and of the concrete of the confining elements. In calculating the shear resistance of the masonry the rules for unreinforced masonry walls subjected to shear loading should be used, considering the length of the masonry element. Reinforcement of confining elements should not be taken into account.

7.11.5.3 In the verification of confined masonry members subjected to lateral loading, the assumptions set out for unreinforced and reinforced masonry walls should be used. The contribution of the reinforcement of the confining elements should be considered.

7.11.6 Confined Masonry Members

7.11.6.1 Confined masonry members shall not exhibit flexural cracking nor deflect excessively under serviceability loading conditions.

7.11.6.2 The verification of confined masonry members at the serviceability limit states shall be based on the assumptions given for unreinforced masonry members.

7.11.7 Architectural Guideline

7.11.7.1 Building Layout

- (a) The building should not be excessively long relative to its width; ideally, the length-to-width ratio should not exceed 4.
- (b) The walls should be continuous up the building height.
- (c) Openings (doors and windows) should be placed in the same position up the building height.

7.11.7.2 Walls

- (a) At least two fully confined walls should be provided in each direction.
- (b) For Seismic Zone 1 and 2, wall density of at least 2% in each of two orthogonal directions is required to ensure good earthquake performance of confined masonry construction. The wall density for Seismic Zones 3 and 4 should be at least 4% and 5% respectively. Wall density can be defined as the total cross sectional area of all walls in one direction divided by the sum of the floor plan areas for all floors in a building.

7.11.7.3 Building Height

Confined masonry is suitable for low- to medium-rise building construction. Confined masonry buildings will be subject to the following height restrictions:

- (a) Up to 4-storey high for Seismic Zone 1 and 2
- (b) Up to 3-storey high for Seismic Zone 3
- (c) Up to 2-storey high for Seismic Zone 4

7.11.8 Confined Masonry Details

7.11.8.1 Confined masonry walls shall be provided with vertical and horizontal reinforced concrete or reinforced masonry confining elements so that they act together as a single structural member.

7.11.8.2 Top and sides confining elements shall be cast after the masonry has been built so that they will be duly anchored together.

7.11.8.3 Vertical confining elements should be placed:

- (a) at the free edges of each structural wall element;
- (b) at both sides of any wall opening with an area of more than 1.5 m^2 ;
- (c) within the wall if necessary in order not to exceed a spacing of 5 m between the confining elements;
- (d) at the intersections of structural walls, wherever the confining elements imposed by the above rules are at a distance larger than 1.5 m.

7.11.8.4 Horizontal confining elements shall be placed in the plane of the wall at every floor level and in any case with a vertical spacing of not more than 4 m.

- 7.11.8.5 Confining elements should have a cross-sectional area not less than 0.02 m^2 , with a minimum dimension of 150 mm in the plan of the wall. In double-leaf walls the thickness of confining elements should assure the connection of the two leaves and their effective confinement.
- 7.11.8.6 The longitudinal reinforcement of confining elements may not have a cross-sectional area less than 300 mm^2 , nor than 1% of the cross-sectional area of the confining element. The detailing of the reinforcements should be in accordance with Chapter 8.
- 7.11.8.7 Stirrups not less than 6 mm in diameter and spaced not more than 300 mm should be provided around the longitudinal reinforcement. Column ties should preferably have 135° hooks – the use of 90° hooks is not recommended. At a minimum, 6 mm ties at 200 mm spacing (6 mm@200 mm) should be provided. It is recommended to use 6 mm ties at 100 mm spacing (6 mm@100 mm) in the column end-zones (top and bottom).
- 7.11.8.8 To ensure the effectiveness of tie-beams in resisting earthquake loads, longitudinal bars should have a 90° hooked anchorage at the intersections, as shown in Figure 6.7.14. The hook length should be at least 500 mm.

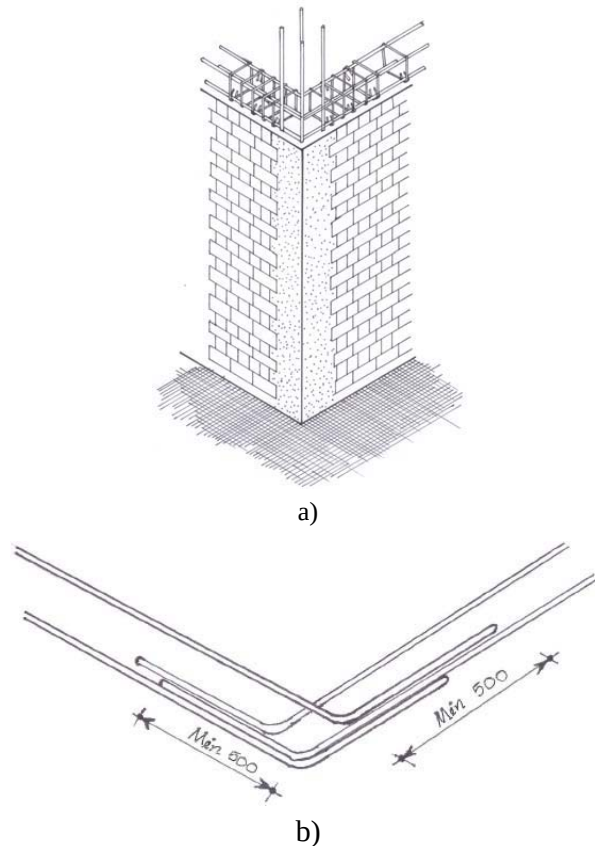


**Fig. 6.4.14: Tie-beam construction: a) wall intersections;
b) hooked anchorage to longitudinal reinforcement**

- 7.11.8.9 Proper detailing of the tie-beam-to-tie-column connections is a must for satisfactory earthquake performance of the entire building. Reinforcing bars must be properly anchored. A typical connection detail at the roof level is shown in Figure 6.7.15. Note that the tie-column reinforcement needs to be extended into the tie-beam as much as possible, preferably up to the underside of the top tie-beam reinforcement. A hooked anchorage needs to be provided (90° hooks) both for the tie-column and tie-beam reinforcement.

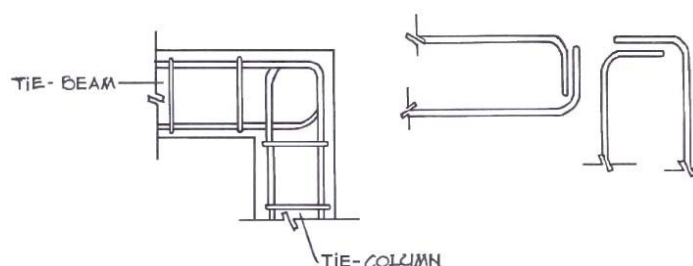


Fig. 6.7.15: Detailing requirement for the tie-beam-to-tie-column connection

7.11.8.10 Special lintel beams may be required across larger openings having a width exceeding 1.5 m. Additional reinforcement bars need to be provided. Lintel beams can be integrated with the tie-beams at the floor level.

7.11.8.11 Lap splices may not be less than 60 bar diameters or 500mm in length. Splicing should take place at column mid height, except for the ground floor level (where splicing is not permitted).

7.11.8.12 The minimum wall thickness should not be less than 100 mm. The wall height/thickness ratio should not exceed 30.

7.11.8.13 Toothed edges should be left on each side of the wall, as shown in Figure 6.7.16 a. Toothed edges are essential for adequate wall confinement, which contributes to satisfactory earthquake performance. Alternatively, when the interface between the masonry wall and the concrete tie-column needs to remain smooth for appearance's sake, steel dowels should be provided in mortar bed joints to ensure interaction between the masonry and the concrete during an earthquake (Figure 6.7.16 b).

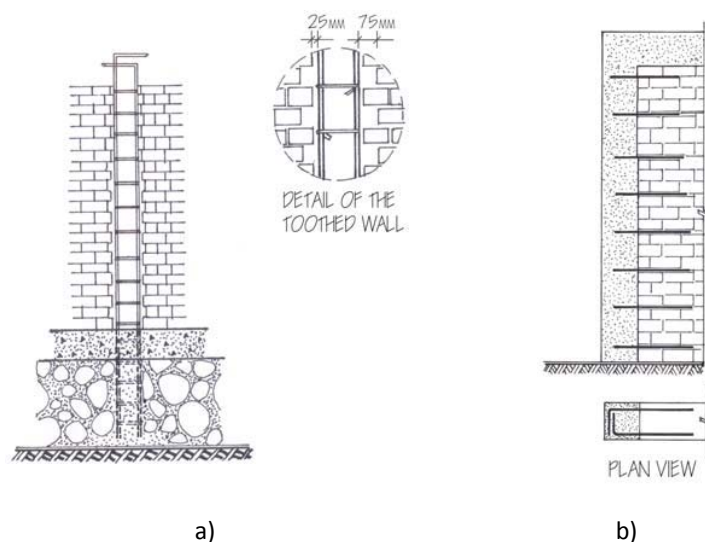


Fig. 6.7.16: a) Toothed wall construction; b) horizontal dowels at the wall-to-column interface

7.11.8.14 Concrete in the tie-columns can be poured once the desired wall height has been reached. The masonry walls provide formwork for the tie-columns on two sides; however the formwork must be placed on the remaining two sides.

7.11.9 Foundation and Plinth Construction

The foundation should be constructed as in traditional brick masonry construction. Either an uncoursed random rubble stone masonry footing or a RC strip footing can be used. A RC plinth band should be constructed on top

of the foundation. In confined masonry construction, plinth band is essential for preventing building settlements in soft soil areas. An alternative foundation solution with RC strip footing is also illustrated in Figure 6.7.17.

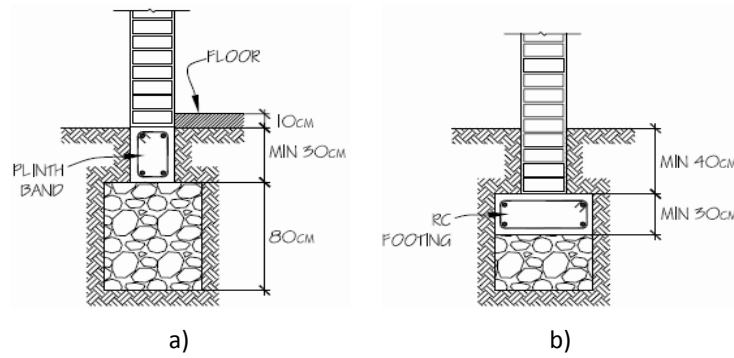


Fig. 6.7.17: Foundation construction: a) RC plinth band and stone masonry foundation; b) RC strip footing

Detailing of Reinforcement in Concrete Structures

8.1 Scope

Provisions of Sec. 8.1 and 8.2 of Chapter 8 shall apply for detailing of reinforcement in reinforced concrete members, in general. For reinforced concrete structures, subject to earthquake loadings in seismic design categories B, C and D, special provisions contained in Sec. 8.3 of this chapter shall apply. For notations used in Sec. 8.1 and 8.2 and not explained therein, see 6.1.1.1.

8.1.1 Standard Hooks

The term "standard hook" as used in this Code shall mean one of the following:

- (a) 180° bend plus an extension of at least 4 bar diameters, but not less than 65 mm at the free end of the bar.
- (b) 90° bend plus an extension of at least 12 bar diameters at the free end of the bar.
- (c) For stirrup and tie anchorage
 - i. For 16 mm ϕ bar and smaller, a 90° bend plus an extension of at least 6 bar diameters at the free end of the bar,
 - ii. For 19 mm to 25 mm ϕ bars, a 90° bend plus an extension of at least 12 bar diameters at the free end of the bar,
 - iii. For 25 mm ϕ bar and smaller, a 135° bend plus an extension of at least 6 bar diameters at the free end of the bar,
 - iv. For closed ties and continuously wound ties, a 135° bend plus an extension of at least 6 bar diameters, but not less than 75 mm.
- (d) Seismic hooks as defined in 2.2.

8.1.2 Minimum Bend Diameters

- 8.1.2.1 The minimum diameter of bend measured on the inside of the bar, for standard hooks other than for stirrups and ties in sizes 10 mm ϕ through 16 mm ϕ , shall not be less than the values shown in Table 8.1.1.

Table 8.1.1 Minimum Diameters of Bend

Bar Size	Minimum Diameter of Bend
$10 \text{ mm} \leq d_b \leq 25 \text{ mm}$	$6d_b$
$25 \text{ mm} < d_b \leq 40 \text{ mm}$	$8d_b$
$40 \text{ mm} < d_b \leq 57 \text{ mm}$	$10d_b$

- 8.1.2.2 For stirrups and tie hooks, inside diameter of bend shall not be less than 4 bar diameters for 16 mm ϕ bar and smaller. For bars larger than 16 mm ϕ , diameter of bend shall be in accordance with Table 8.1.1.
- 8.1.2.3 Inside diameter of bend in welded wire reinforcement for stirrups and ties shall not be less than 4 bar diameters for deformed wire larger than MD40 and 2 bar diameters for all other wires. Bends with inside diameter of less than 8 bar diameters shall not be less than 4 bar diameters from nearest welded intersection.

8.1.3 Bending

- 8.1.3.1 Unless otherwise permitted by the engineer, all reinforcement shall be bent cold.
- 8.1.3.2 Reinforcement partially embedded in concrete shall not be bent in place, except as permitted by the engineer or as shown in the design drawings.

8.1.4 Surface Conditions of Reinforcement

- 8.1.4.1 When concrete is placed, metal reinforcement shall be free from mud, oil, or other nonmetallic coatings that decrease bond. Epoxy-coating of steel reinforcement in accordance with standards referenced in this code shall be permitted.
- 8.1.4.2 Metal reinforcement with rust, mill scale, or a combination of both, shall be considered satisfactory, provided the minimum dimensions (including height of deformations) and weight of a hand-wire-brushed test specimen are not less than applicable ASTM specification requirements.

8.1.5 Placing of Reinforcement

- 8.1.5.1 Reinforcement shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in 8.1.5.2 below.
- 8.1.5.2 Reinforcement shall be placed within the following tolerances unless otherwise specified by the engineer:
- (a) Tolerances for depth d , and minimum concrete cover in flexural members, walls and compression members shall be as set forth in Table 8.1.2.

Table 8.1.2 Tolerances for Placing Reinforcement

	Tolerance for d	Tolerance for Minimum Concrete Cover
$d \leq 200$ mm	± 10 mm	-10 mm
$d > 200$ mm	± 13 mm	-13 mm

- (b) Notwithstanding the provision of (a) above, 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 specified in the design drawings or specifications.
- (c) Tolerance for longitudinal location of bends and ends of reinforcement shall be ± 50 mm, except at discontinuous ends of brackets and corbels, where tolerance shall be ± 13 mm and at discontinuous ends of other members, where tolerance shall be ± 25 mm. The tolerance for concrete cover of 8.1.5.2a shall also apply at discontinuous ends of members.
- 8.1.5.3 Welded wire reinforcement (with wire size not greater than MW30 or MD30) used in slabs not exceeding 3 m in span shall be permitted to be curved from a point near the top of slab over the support to a point near the bottom of slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support.
- 8.1.5.4 Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the engineer.

8.1.6 Spacing of Reinforcement

- 8.1.6.1 The minimum clear spacing between parallel bars in a layer shall be equal to one bar diameter, but not less than 25 mm, or 4/3 the maximum nominal size of coarse aggregate, whichever is larger.

- 8.1.6.2 Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above those in the bottom layer with clear distance between layers not less than 25 mm.
- 8.1.6.3 For compression members, the clear distance between longitudinal bars shall be not less than 1.5 bar diameters nor 40 mm nor $4/3$ the maximum nominal size of coarse aggregate.
- 8.1.6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.
- 8.1.6.5 In walls and one-way slabs the maximum bar spacing shall not be more than three times the wall or slab thickness h nor 450 mm.
- 8.1.6.6 For two-way slabs, maximum spacing of bars shall not exceed two times the slab thickness h nor 450 mm.
- 8.1.6.7 For temperature steel only, maximum spacing shall not exceed five times the slab thickness h nor 450 mm.
- 8.1.6.8 **Bundled Bars**
- (a) Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.
 - (b) Bundled bars shall be enclosed within stirrups or ties.
 - (c) Bars larger than 32 mm ϕ shall not be bundled in beams.
 - (d) Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.
 - (e) 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 total area.

8.1.7 Exposure Condition and Cover to Reinforcement

- 8.1.7.1 The nominal concrete cover to all reinforcement (including links), maximum free water-cement ratio and minimum cement content required for various minimum concrete strengths used in different exposure conditions shall be as specified in Table 8.1.3. However, for mild environment, the minimum concrete cover specified in 8.1.7.2 and 8.1.7.3 for various structural elements may be used.

8.1.7.2 Cast-in-place Concrete :

- (a) Minimum concrete cover for concrete cast against and permanently exposed to earth shall be 75 mm.
- (b) Concrete exposed to earth or weather:

	Minimum cover, mm
19 mm ϕ through 57 mm ϕ	50
16 mm ϕ bar and smaller	40

- (c) The following minimum concrete cover may be provided for reinforcement for concrete surfaces not exposed to weather or in contact with ground:

	Minimum cover, mm
Slabs, Walls :	
40 mm ϕ to 57 mm ϕ	40
36 mm ϕ bar and smaller	20

Minimum cover, mm

Beams, Columns :

Primary reinforcement, Ties, stirrups, spirals 40

Shells, folded plate members :

19 mm ϕ bar and larger 20

16mm ϕ bar and smaller 16

Table 8.1.3* Concrete Cover and other Requirements for Various Exposure Conditions

Environment		Exposure Conditions	Minimum f'_c N/mm ²						
Nominal cover (mm)			20	25	30	35	40	45	50
	Mild	Concrete surfaces protected against weather or aggressive conditions	30	25	20	20	20**	20**	20**
	Moderate	Concrete surface away from severe rain Concrete subject to condensation Concrete surfaces continuously under water Concrete in contact with non-aggressive soil	40	35	30	25	20	20	20
	Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or severe condensation		45	40	30	25	25	20
	Very severe	Concrete surfaces exposed to sea water spray, corrosive fumes			50	40	30	30	25
	Extreme	Concrete surfaces exposed to abrasive action, e.g. sea water carrying solids or flowing water with pH $\leq$ 4.5 or machinery or vehicles				60	50	40	30
Maximum water/cement ratio			0.65	0.65	0.60	0.55	0.50	0.45	0.42
Minimum cement content, (kg/m ³)			315	325	350	375	400	410	420

*This table relates to aggregate of 20 mm nominal maximum size.

** May be reduced to 15 mm provided the nominal maximum aggregate size does not exceed 15mm.

8.1.7.3 Precast Concrete (manufactured under plant control conditions) :

(a) Concrete exposed to earth or weather:

Minimum cover, mm

Wall Panels:

40 mm ϕ to 57 mm ϕ 40

36 mm ϕ bar and smaller 20

Other Members:

40 mm ϕ to 57 mm ϕ 50

19 mm ϕ through 36 mm ϕ 40

16mm ϕ bar and smaller 30

(b) Concrete not exposed to weather or in contact with ground:

	Minimum cover, mm
Slabs, Walls:	
40 mm ϕ to 57 mm ϕ	30
36 mm ϕ bar and smaller	16
Beams, columns :	
Primary reinforcement	$20 \leq d_b \leq 40$
Ties, stirrups, spiral	15
Shells, folded plate members :	
19 mm ϕ bar and larger	16
16 mm ϕ bar and smaller	10

8.1.7.4 For concrete cast against and permanently exposed to earth, minimum cover shall be 75 mm. If, concrete cover specified in 8.1.7.1 (Table 8.1.3) conflicts with those specified in 8.1.7.2 or 8.1.7.3, the larger value shall be taken.

8.1.7.5 **Bundled Bars** : Minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 50 mm.

8.1.7.6 **Future Extension** : Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

8.1.7.7 **Fire Protection** : If a thickness of cover for fire protection greater than the concrete covers specified in 8.1.7.1 through 8.1.7.6 is required, such greater thicknesses shall be specified.

8.1.7.8 **Corrosive Environments**: If a thickness of cover for corrosive environment or other severe exposure conditions greater than the concrete covers specified in 8.1.7.1 through 8.1.7.6 is required, such greater thicknesses shall be specified. For corrosion protection, a specified concrete cover for reinforcement not less than 50 mm for walls and slabs and not less than 65 mm for other members may be used. For precast concrete members a specified concrete cover not less than 40 mm for walls and slabs and not less than 50 mm for other members may be used.

8.1.8 Reinforcement Details for Columns

8.1.8.1 **Offset Bars** : Offset bent longitudinal bars shall conform to the following:

- The maximum slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.
- Portions of bar above and below an offset shall be parallel to the axis of column.
- 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 1.5 times the horizontal component of the computed force in the inclined portion of the offset bars. Lateral ties or spirals, if used, shall be placed not more than 150 mm away from points of bend.
- Offset bars shall be bent before placement in the forms (see 8.1.3).
- Where the face of the column above is offset 75 mm or more from the face of the column below, longitudinal bars shall not be permitted to be offset bent. The longitudinal bars adjacent to the offset column faces shall be lap spliced using separate dowels. Lap splices shall conform to 8.2.14.

8.1.8.2 **Steel Cores**: Load transfer in structural steel cores of composite compression members shall be provided by the following:

- 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.

- (b) At end bearing splices, bearing shall be considered effective to transfer not more than 50 per cent of the total compressive stress in the steel core.
- (c) Transfer of stress between column base and footing shall be designed in accordance with 6.8.8.
- (d) Base of structural steel section shall be designed to transfer the total load from the entire composite member to the footing; or, the base shall 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.

8.1.9 Lateral Reinforcement for Columns

8.1.9.1 Lateral reinforcement for compression members shall conform to the provisions of 8.1.9.3 and 8.1.9.4 below and where shear or torsion reinforcement is required, shall also conform to provisions of Sec 6.4.

8.1.9.2 Lateral reinforcement requirements for composite columns shall conform to 6.3.13.7 and 6.3.13.8.

8.1.9.3 **Spirals :** Spiral reinforcement for columns shall conform to 6.3.9.3 and to the following:

- (a) Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled as to permit handling and placing without distortion from designed dimensions.
- (b) Size of spirals shall not be less than 10 mm diameter for cast-in-place construction.
- (c) The minimum and maximum clear spacing between spirals shall be 25 mm and 75 mm respectively.
- (d) Anchorage of spiral reinforcement shall be provided by 1.5 extra turns of spiral bar or wire at each end of a spiral unit.
- (e) Splices in spiral reinforcement shall be lap splices of 48 spiral diameter for deformed uncoated bar or wire and 72 spiral diameter for other cases, but not less than 300 mm.
- (f) Spirals shall extend from the top of footing or slab in any storey to the level of the lowest horizontal reinforcement in members supported above.
- (g) Spirals shall extend above termination of spiral to bottom of slab or drop panel, where beams or brackets do not frame into all sides of a column.
- (h) Spirals shall extend to a level at which the diameter or width of capital is 2 times that of the column, in case of columns with capitals.
- (i) Spirals shall be held firmly in place and true to line.

8.1.9.4 **Ties :** Tie reinforcement for compression members shall conform to the following:

- (a) All bars shall be enclosed by lateral ties, at least 10 mm ϕ in size for longitudinal bars 32 mm ϕ or smaller, and at least 12 mm ϕ in size for 36 mm ϕ to 57 mm ϕ and bundled longitudinal bars.
- (b) Vertical spacing of ties shall not exceed 16 longitudinal bar diameters or 48 tie diameters, or the least dimension of the compression members.
- (c) 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 not more than 135 deg. No vertical 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 is allowed.
- (d) The lowest tie in any storey shall be placed within one-half the required tie spacing from the top most horizontal reinforcement in the slab or footing below. The uppermost tie in any storey shall be within one-half the required tie spacing from the lowest horizontal reinforcement in the slab or drop panel above.
- (e) Where beams or brackets provide concrete confinement at the top of the column on all (four) sides, the top tie shall be within 75 mm of the lowest horizontal reinforcement in the shallowest of such beams or brackets.
- (f) Where anchor bolts are placed in the top of columns or pedestals, the bolts shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 125 mm of the top of the column or pedestal, and shall consist of at least two 12 mm ϕ bars or three 10 mm ϕ bars.
- (g) Where longitudinal bars are arranged in a circular pattern, individual circular ties per specified spacing may be used.

8.1.10 Lateral Reinforcement for Beams

- 8.1.10.1 Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in 8.1.9.4 above. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.
- 8.1.10.2 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.
- 8.1.10.3 Closed ties or stirrups shall 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 B splice (lap of $1.3\ell_d$) or anchored in accordance with 8.2.10.

8.1.11 Shrinkage and Temperature Reinforcement

- 8.1.11.1 Where the flexural reinforcement extends in one direction only, reinforcement for shrinkage and temperature stresses shall be provided perpendicular to flexural reinforcement in structural slabs. Shrinkage and temperature reinforcement shall be provided in accordance with 8.1.11.2 below.
- 8.1.11.2 Deformed reinforcement conforming to 5.3.2 shall be provided in accordance with the following:
- Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area:

0.0020	for slabs where reinforcement with $f_y = 275 \text{ N/mm}^2$ or 350 N/mm^2 are used,
0.0018	for slabs where reinforcement with $f_y = 420 \text{ N/mm}^2$ are used,
$0.0018 \left(\frac{420}{f_y} \right)$	or slabs where reinforcement with f_y exceeding 420 N/mm^2 are used.

In any case, the reinforcement ratio shall not be less than 0.0014.
 - Area of shrinkage and temperature reinforcement for brick aggregate concrete shall be at least 1.5 times that provided in (a) above.
 - Shrinkage and temperature reinforcement shall be spaced not farther apart than 5 times the slab thickness, nor 450 mm.
 - At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the specified yield strength f_y in tension in accordance with Sec 8.2.

8.1.12 Requirements for Structural Integrity

- 8.1.12.1 In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure.
- 8.1.12.2 The minimum requirements for cast-in-place construction shall be:
- In one-way slab construction, at least one bottom bar shall be continuous or shall be spliced over the support with a Class A tension splice. At non-continuous supports, the bars may be terminated with a standard hook.
 - Beams at the perimeter of the structure shall have at least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars and one-quarter of the positive moment reinforcement required at midspan, but not less than two bars made continuous over the span length passing through the region bounded by the longitudinal reinforcement of the column around the perimeter and tied with closed stirrups. Closed stirrups need not be extended through any joints. The required continuity may be provided with top reinforcement spliced at mid-span and bottom reinforcement spliced at or near the support with Class B tension splices.
 - When closed stirrups are not provided, in other than perimeter beams, at least one-quarter of the positive moment reinforcement required at mid-span, but not less than two bars shall pass through the region bounded by the longitudinal reinforcement of the column and shall be continuous or shall

be spliced over the support with a Class B tension splice. At non-continuous supports the bars shall be anchored to develop f_y at the face of the support using a standard hook.

- 8.1.12.3 To effectively tie elements together, tension ties shall be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure for precast concrete construction.

8.1.13 Connections

- 8.1.13.1 Enclosure shall be provided for splices of continuing reinforcement and for anchorage of terminating reinforcement at connections of principal framing elements (such as beams and columns),
- 8.1.13.2 External concrete or internal closed ties, spirals, or stirrups shall be used as enclosures at connections.

8.2 DEVELOPMENT AND SPLICES OF REINFORCEMENT

8.2.1 Development of Reinforcement - General

Calculated tension or compression stress 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 thereof. Hooks may be used in developing bars in tension only.

8.2.2 Scope and Limitation

The values of $\sqrt{f'_c}$ used in Sec 8.2 shall not exceed 8.3 MPa. In addition to requirements stated here that affect detailing of reinforcement, structural integrity requirements of 8.1.12 shall be satisfied.

8.2.3 Development of Deformed Bars and Deformed Wires in Tension

- 8.2.3.1 Development length for deformed bars and deformed wire in tension, l_d shall be determined from either 8.2.3.2 or 8.2.3.3 and applicable modification factors of 8.2.3.4 and 8.2.3.5, but l_d shall not be less than 300 mm.

- 8.2.3.2 For deformed bars or deformed wire, l_d shall be as follows:

Spacing and cover	19 mm ϕ and smaller bars and deformed wires	20 mm ϕ and larger bars
Clear spacing of bars or wires being developed or spliced not less than d_b , clear cover not less than d_b , and stirrups or ties throughout l_d not less than the Code minimum or Clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\frac{f_y \psi_t \psi_e}{2.1 \lambda \sqrt{f'_c}} \right) d_b$	$\left(\frac{f_y \psi_t \psi_e}{1.7 \lambda \sqrt{f'_c}} \right) d_b$
Other cases	$\left(\frac{f_y \psi_t \psi_e}{1.4 \lambda \sqrt{f'_c}} \right) d_b$	$\left(\frac{f_y \psi_t \psi_e}{1.1 \lambda \sqrt{f'_c}} \right) d_b$

- 8.2.3.3 For deformed bars or deformed wire, l_d shall be

$$l_d = \left(\frac{f_y}{1.1 \lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \quad 8.2.1$$

in which the confinement term $(c_b + K_{tr})/d_b$ shall not be taken greater than 2.5, and

$$K_{tr} = \frac{40 A_{tr}}{sn} \quad 8.2.2$$

where n is the number of bars or wires being spliced or developed along the plane of splitting. It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present.

8.2.3.4 The factors used in the expressions for development of deformed bars and deformed wires in tension in 8.2.3 are as follows:

- Where horizontal reinforcement is placed such that more than 300 mm of fresh concrete is cast below the development length or splice, $\psi_t = 1.3$. For other situations, $\psi_t = 1.0$.
- For epoxy-coated bars or wires with cover less than $3d_b$, or clear spacing less than $6d_b$, $\psi_e = 1.5$. For all other epoxy-coated bars or wires, $\psi_e = 1.2$. For uncoated and zinc-coated (galvanized) reinforcement, $\psi_e = 1.0$. However, the product $\psi_t \psi_e$ need not be greater than 1.7.
- For 19 mm ϕ and smaller bars and deformed wires, $\psi_s = 0.8$. For 20 mm ϕ and larger bars, $\psi_s = 1.0$.
- Where lightweight concrete is used, λ shall not exceed 0.75 unless f_{ct} is specified (see 6.1.8.1). Where normalweight concrete is used, $\lambda = 1.0$.

8.2.3.5 **Excess Reinforcement** : Development length may be reduced by the factor $\left[\frac{A_{s,required}}{A_{s,Provided}} \right]$ where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for f_y is specifically required or the reinforcement is designed under the provisions of 8.3.3.1(c).

8.2.4 Development of Deformed Bars and Deformed Wires in Compression

8.2.4.1 Development length for deformed bars and deformed wire in compression, l_{dc} , shall be determined from 8.2.4.2 and applicable modification factors of 8.2.4.3, but l_{dc} shall not be less than 200 mm.

8.2.4.2 For deformed bars and deformed wire, l_{dc} shall be taken as the larger of $(0.24f_y/\lambda\sqrt{f'_c})d_b$ and $(0.043f_y)d_b$, with λ as given in 8.2.3.4(d) and the constant 0.043 carries the unit of mm^2/N .

8.2.4.3 Length l_{dc} in 8.2.4.2 shall be permitted to be multiplied by the applicable factors for:

- Reinforcement in excess of that required by analysis $\left[\frac{A_{s,required}}{A_{s,Provided}} \right]$
- Reinforcement enclosed within spiral reinforcement not less than 6 mm diameter and not more than 100 mm pitch or within 12 mm ϕ ties in conformance with 8.1.9.4 and spaced at not more than 100 mm on center 0.75

8.2.5 Development of Bundled Bars

8.2.5.1 Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20 per cent for 3 bar bundles and 33 per cent for 4 bar bundles.

8.2.5.2 For determining the appropriate spacing and cover values in 8.2.3.2, the confinement term in 8.2.3.3, and the ψ_e factor in 8.2.3.4(b), a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area and having a centroid that coincides with that of the bundled bars.

8.2.6 Development of Standard Hooks in Tension

8.2.6.1 Development length l_{dh} for deformed bars in tension terminating in a standard hook shall be computed as the product of the basic development length for deformed bars, l_{dh} of 8.2.6.2 below and the applicable modification factor or factors of 8.2.6.3, but l_{dh} shall be not less than $8d_b$, nor less than 150 mm.

8.2.6.2 For deformed bars, l_{dh} shall be $(0.24\psi_e f_y / \lambda \sqrt{f'_c}) d_b$ with ψ_e taken as 1.2 for epoxy-coated reinforcement, and λ taken as 0.75 for lightweight concrete. For other cases, ψ_e and λ shall be taken as 1.0.

8.2.6.3 Length l_{dh} in 8.2.6.2 shall be permitted to be multiplied by the following applicable factors:

(a) For 36 mm ϕ bar and smaller hooks with side cover (normal to plane of hook) not less than 65 mm, and for 90-degree hook with cover on bar extension beyond hook not less than 50 mm 0.7

(b) For 90-degree hooks of 36 mm ϕ bar and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along l_{dh} ; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend 0.8

(c) For 180-degree hooks of 36 mm ϕ bar and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along l_{dh} 0.8

(d) Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis $\left[\frac{A_{s,required}}{A_{s,provided}} \right]$

In 8.2.6.3(b) and 8.2.6.3(c), d_b is the diameter of the hooked bar, and the first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend.

8.2.6.4 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 65 mm, the hooked bar shall be enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along l_{dh} . The first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend, where d_b is the diameter of the hooked bar. For this case, the factors of 8.2.6.3(b) and (c) shall not apply.

8.2.6.5 Hooks shall not be considered effective in developing bars in compression.

8.2.7 Development of Flexural Reinforcement - General

8.2.7.1 Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

8.2.7.2 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. In addition, the provisions of 8.2.8.3 shall also be satisfied.

8.2.7.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance not less than d nor less than $12d_b$, except at supports of simple spans and at free end of cantilevers.

8.2.7.4 Continuing reinforcement shall have an embedment length not less than the development length l_d beyond the point where the bent or terminated tension reinforcement is no longer needed to resist bending.

8.2.7.5 No flexural bar shall be terminated in a tension zone unless one of the following conditions is satisfied:

(a) V_u at the location of termination is not over two-thirds of ϕV_n .

(b) Stirrup area in excess of that normally required for shear and torsion is provided over a distance along each terminated bar or wire equal to $0.75d$ from the point of cutoff. Excess stirrup area A_v shall be not less than $\frac{0.41b_w s}{f_{yt}}$. Spacing s shall not exceed $\frac{d}{8\beta_b}$, where β_b is the ratio of area of reinforcement cut off to total area of tension reinforcement at the section.

- (c) For 36 mm ϕ bar and smaller, the continuing bars provide twice the area required for flexure at the cutoff point and the shear V_u does not exceed three-quarter of ϕV_n .

8.2.7.6 Where the reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered footings, brackets, deep flexural members, or members in which tension reinforcement is not parallel to the compression face, adequate anchorage shall be provided for the tension reinforcement. See 8.2.8.4 and 8.2.9.4 for deep flexural members.

8.2.8 Development of Positive Moment Reinforcement

8.2.8.1 At least one-third of the positive moment reinforcement in simple members and one-fourth of the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm.

8.2.8.2 When the flexural member is a part of the primary lateral load resisting system, positive moment reinforcement extended into the support by 8.2.8.1 above shall be anchored to develop the specified yield strength f_y in tension at the face of support.

8.2.8.3 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 8.2.3 satisfies Eq (8.2.3), except that Eq (8.2.3) need not be satisfied for reinforcement terminating beyond the centreline of simple supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook.

$$l_d \leq \frac{M_n}{V_u} + l_a \quad (8.2.3)$$

where

M_n = nominal moment strength assuming all reinforcement at section to be stressed to f_y .

V_u = factored shear force at section

l_a = at a support, embedded length of bar beyond centre of support; at point of zero moment, shall be limited to d or $12 d_b$, whichever is greater.

The value of M_n/V_u may be increased 30 per cent when the ends of reinforcement are confined by a compressive reaction.

8.2.8.4 At simple supports of deep beams, positive moment tension reinforcement shall be anchored to develop f_y in tension at the face of the support except that if design is carried out using Appendix A, the positive moment tension reinforcement shall be anchored in accordance with A.4.3. At interior supports of deep beams, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

8.2.9 Development of Negative Moment Reinforcement

8.2.9.1 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.

8.2.9.2 Negative moment reinforcement shall have an embedment length into the span as required by 8.2.1, 8.2.2. and 8.2.7.3.

8.2.9.3 At least one-third of the total tension reinforcement provided for negative moment at the support shall be extended beyond the point of inflection a distance not less than d , $l_n/16$, or $12d_b$, whichever is greater.

- 8.2.9.4 At interior supports of deep flexural members, negative moment tension reinforcement shall be continuous with that of the adjacent spans.

8.2.10 Development of Shear Reinforcement

- 8.2.10.1 Shear reinforcement shall be carried as close to compression and tension surfaces of member as cover requirements and proximity of other reinforcement permits.
- 8.2.10.2 The ends of single leg, simple U-, or multiple U-stirrups shall be anchored by one of the following means:
- (a) By a standard hook around longitudinal reinforcement for MD200 wires, and 16 mm ϕ bars and smaller and for 19 mm ϕ through 25 mm ϕ bars with $f_{yt} \leq 280 \text{ N/mm}^2$.
 - (b) For 19 mm ϕ through 25 mm ϕ stirrups with f_{yt} greater than 280 N/mm^2 , a standard stirrup hook around a longitudinal bar plus an embedment between mid-height of the member and the outside end of the hook equal to or greater than $\frac{0.17d_b f_{yt}}{\lambda \sqrt{f'_c}}$.
 - (c) For each leg of welded plain wire reinforcement forming simple U-stirrups, either: (i) Two longitudinal wires spaced at a 50 mm spacing along the member at the top of the U; or (ii) 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 shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.
 - (d) For each end of a single leg stirrup of welded wire reinforcement, 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 $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.
 - (e) In joist construction, for 13 mm ϕ bar and MD130 wire and smaller, a standard hook.
- 8.2.10.3 Each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar between anchored ends.
- 8.2.10.4 If extended into the region of tension, longitudinal bars bent to act as shear reinforcement 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 8.2.3 for that part of f_{yt} required to satisfy Eq (6.4.12).
- 8.2.10.5 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are $1.3l_d$. In members at least 450 mm deep, such splices with $A_b f_{yt}$ not more than 40 kN per leg shall be considered adequate if stirrup legs extend the full available depth of member.

8.2.11 Development of Plain Bars

For plain bars, the minimum development length shall be twice that of deformed bars specified in 8.2.1 through 8.2.10 above.

8.2.12 Splices of Reinforcement - General

- 8.2.12.1 Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the engineer.
- 8.2.12.2 **Lap Splices**
- (a) Lap splices shall not be used for 36 mm ϕ bars and larger, except as provided in 8.2.14.2 & 6.8.8.2.3.
 - (b) Lap splices of bundled bars shall be based on the lap splice length required for individual bars within the bundle, increased in accordance with 8.2.5. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

- (c) Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, nor 150 mm.

8.2.12.3 Welded Splices and Mechanical Connections

- (a) Welded splices and other mechanical connections are allowed.
 (b) Except as provided in this Code, all welding shall conform to "Structural Welding Code - Reinforcing Steel" (AWS D1.4).
 (c) Welded splices shall be butted and welded to develop in tension at least 125 per cent of specified yield strength f_y of the bar.
 (d) A full mechanical connection shall develop in tension or compression, as required, at least 125 per cent of specified yield strength f_y of the bar.
 (e) Welded splices and mechanical connections not meeting the requirements of (c) or (d) above are allowed only for 16 mm ϕ bars or smaller and in accordance with 8.2.13.4.

8.2.13 Splices of Deformed Bars and Deformed Wire in Tension

- 8.2.13.1 The minimum length of lap for tension splices shall be as required for Class A or B splice, but not less than 300 mm, where the classification shall be as follows:

Class A splice $1.0 l_d$

Class B splice $1.3 l_d$

Where l_d is calculated in accordance with 8.2.3 to develop f_y , but without the 300 mm minimum of 8.2.3.1 and without the modification factor of 8.2.3.5.

- 8.2.13.2 Lap splices of deformed bars and deformed wire in tension shall be class B splices except that Class A splices are allowed when the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice, and one-half or less of the total reinforcement is spliced within the required lap length.

- 8.2.13.3 Where area of reinforcement provided is less than twice that required by analysis, welded splices or mechanical connections used shall meet the requirements of 8.2.12.3(c) or 8.2.12.3(d) above.

- 8.2.13.4 Welded splices or mechanical connections not meeting the requirements of 8.2.12.3(c) or 8.2.12.3(d) shall be permitted for 16 mm ϕ bars or smaller if the following requirements are met :

- (a) 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 the section but not less than 140 N/mm^2 for total area of reinforcement provided.
 (b) Spliced reinforcement stress shall be taken as the specified splice strength, in computing tensile force developed at each section, but not to exceed f_y . Unspliced reinforcement stress shall be taken as a fraction of f_y defined by the ratio of the shortest actual development length provided beyond the section to l_d but not to be taken greater than f_y .

- 8.2.13.5 When bars of different size are lap spliced in tension, splice length shall be the larger of l_d of larger bar and tension lap splice length of smaller bar.

- 8.2.13.6 Splices in tension tie members shall be made with a full welded splice or full mechanical connection in accordance with 8.2.12.3(c) or 8.2.12.3(d) and splices in adjacent bars shall be staggered at least 750 mm.

8.2.14 Splices of Deformed Bars in Compression

- 8.2.14.1 The minimum length of lap for compression splice shall be $0.071 f_y d_b$ for f_y equal to 420 N/mm^2 or less or $(0.13 f_y - 24) d_b$ for f_y greater than 420 N/mm^2 , but not less than 300 mm. For f'_c less than 21 N/mm^2 , length of lap shall be increased by one-third.

8.2.14.2 When bars of different diameters are lap spliced in compression, the splice length shall be the larger of the development length, l_{dc} of the larger bar, and the compression splice length of the smaller bar. Lap splices of 40 mm ϕ , 43 mm ϕ , 50 mm ϕ and 57 mm ϕ bars to 36 mm ϕ and smaller bars shall be permitted.

8.2.14.3 Welded splices or mechanical connections used in compression shall satisfy the requirements of 8.2.12.3(c) or 8.2.12.3(d).

8.2.14.4 End Bearing Splices

- (a) Compression splices for bars required to transmit compressive stress only may consist of end bearing of square cut ends held in concentric contact by a suitable device.
- (b) Bar ends shall terminate in flat surfaces within 1.5 degrees of a right angle to the axis of the bars, and shall be fitted within 3 degrees of full bearing after assembly.
- (c) End bearing splices shall be used only in members containing closed ties, closed stirrups or spirals.

8.2.15 Special Splice Requirements for Columns

8.2.15.1 Lap splices, butt welded splices, mechanical connections, or end-bearing splices shall be used with the limitations of 8.2.15.2 through 8.2.15.4 below. A splice shall satisfy the requirements for all load combinations for the column.

8.2.15.2 Lap Splices in Columns

- (a) Lap splices shall conform to 8.2.14.1, 8.2.14.2, and where applicable to 8.2.15.2(d) or 8.2.15.2(e) below, where the bar stress due to factored loads is compressive.
- (b) Where the bar stress due to factored loads is tensile and does not exceed $0.5f_y$ in tension, lap splices shall be Class B tension lap splices if more than one-half of the bars are spliced at any section, or Class A tension lap splices if half or fewer of the bars are spliced at any section and alternate lap splices are staggered by l_d .
- (c) Where the bar stress due to factored loads is greater than $0.5f_y$ in tension, lap splices shall be Class B tension lap splices.
- (d) In tied reinforced compression members, if throughout lap splice length ties have an effective area of at least $0.0015 h_s$ in both directions, lap splice length is permitted to 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.
- (e) For spirally reinforced compression members, lap splice length of bars within a spiral is permitted to be multiplied by 0.75, but lap length shall not be less than 300 mm.

8.2.15.3 **Welded Splices or Mechanical Connectors in Columns:** Welded splices or mechanical connectors in columns shall meet the requirements of 8.2.12.3(c) or 8.2.12.3(d).

8.2.15.4 **End Bearing Splices in Columns:** End bearing splices complying with 8.2.14.4 may be used for column bars stressed in compression provided the splices are staggered or additional bars are provided at splice locations. The continuing bars in each face of the column shall have a tensile strength at least $0.25f_y$ times the area of the vertical reinforcement in that face.

8.2.16 Splices of Plain Bars

For plain bars, the minimum length of lap shall be twice that of deformed bars specified in 8.2.12 through 8.2.15 above.

8.2.17 Development of headed and mechanically anchored deformed bars in tension

8.2.17.1 Development length for headed deformed bars in tension, l_{dt} , shall be determined from 8.2.17.2. Use of heads to develop deformed bars in tension shall be limited to conditions satisfying (a) through (f):

- (a) Bar f_y shall not exceed 420 MPa;

- (b) Bar size shall not exceed 36 mm ϕ ;
- (c) Concrete shall be normalweight;
- (d) Net bearing area of head A_{brg} shall not be less than $4A_b$;
- (e) Clear cover for bar shall not be less than $2d_b$; and
- (f) Clear spacing between bars shall not be less than $4d_b$.

8.2.17.2 For headed deformed bars, development length in tension l_{dt} shall be $(0.19\psi_e f_y / \sqrt{f'_c}) d_b$, where the value of f'_c used to calculate l_{dt} shall not exceed 40 MPa, and factor ψ_e shall be taken as 1.2 for epoxy-coated reinforcement and 1.0 for other cases. Where reinforcement provided is in excess of that required by analysis, except where development of f_y is specifically required, a factor of $(A_{s,required}) / (A_{s,provided})$ may be applied to the expression for l_{dt} . Length l_{dt} shall not be less than the larger of $8d_b$ and 150 mm.

8.2.17.3 Heads shall not be considered effective in developing bars in compression.

8.2.17.4 Any mechanical attachment or device capable of developing f_y of reinforcement is allowed, provided that test results showing the adequacy of such attachment or device are approved by the Engineer. Development of reinforcement shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the critical section and the mechanical attachment or device.

8.2.18 Development of welded deformed wire reinforcement in tension

8.2.18.1 Development length for welded deformed wire reinforcement in tension, l_d , measured from the point of critical section to the end of wire shall be computed as the product of l_{dl} from 8.2.3.2 or 8.2.3.3, times welded deformed wire reinforcement factor, ψ_w , from 8.2.18.2 or 8.2.18.3. It shall be permitted to reduce l_d in accordance with 8.2.3.5 when applicable, but l_d shall not be less than 200 mm except in computation of lap splices by 8.2.20. When using ψ_w from 8.2.18.2, it shall be permitted to use an epoxy-coating factor ψ_e of 1.0 for epoxy-coated welded deformed wire reinforcement in 8.2.3.2 and 8.2.3.3.

8.2.18.2 For welded deformed wire reinforcement with at least one cross wire within l_d and not less than 50 mm from the point of the critical section, ψ_w shall be the greater of

$$\left(\frac{f_y - 240}{f_y} \right)$$

And

$$\left(\frac{5d_b}{s} \right)$$

but not greater than 1.0, where s is the spacing between the wires to be developed.

8.2.18.3 For welded deformed wire reinforcement with no cross wires within l_d or with a single cross wire less than 50 mm from the point of the critical section, ψ_w shall be taken as 1.0, and l_d shall be determined as for deformed wire.

8.2.18.4 Where any plain wires, or deformed wires larger than D-31, are present in the welded deformed wire reinforcement in the direction of the development length, the reinforcement shall be developed in accordance with 8.2.19.

8.2.19 Development of welded plain wire reinforcement in tension

Yield strength of welded plain wire reinforcement shall be considered developed by embedment of two cross wires with the closer cross wire not less than 50 mm from the point of the critical section. However, l_{dl} shall not be less than

$$l_d = 3.3 \frac{A_b}{s} \frac{f_y}{\lambda \sqrt{f'_c}} \quad (8.2.4)$$

where l_d is measured from the point of the critical section to the outermost crosswire, s is the spacing between the wires to be developed, and λ as given in 8.2.3.4(d). Where reinforcement provided is in excess of that required, l_d may be reduced in accordance with 8.2.3.5. Length, l_d , shall not be less than 150 mm except in computation of lap splices by 8.2.21.

8.2.20 Splices of welded deformed wire reinforcement in tension

- 8.2.20.1 Minimum lap splice length of welded deformed wire reinforcement measured between the ends of each reinforcement sheet shall be not less than the larger of $1.3l_d$ and 200 mm, and the overlap measured between outermost cross wires of each reinforcement sheet shall be not less than 50 mm, where l_d is calculated in accordance with 8.2.18 to develop f_y .
- 8.2.20.2 Lap splices of welded deformed wire reinforcement, with no cross wires within the lap splice length, shall be determined as for deformed wire.
- 8.2.20.3 Where any plain wires, or deformed wires larger than MD200, are present in the welded deformed wire reinforcement in the direction of the lap splice or where welded deformed wire reinforcement is lap spliced to welded plain wire reinforcement, the reinforcement shall be lap spliced in accordance with 8.2.21.

8.2.21 Splices of welded plain wire reinforcement in tension

Minimum length of lap for lap splices of welded plain wire reinforcement shall be in accordance with 8.2.21.1 and 8.2.21.2.

- 8.2.21.1 Where A_s provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each reinforcement sheet shall be not less than the largest of one spacing of cross wires plus 50 mm, $1.5 l_d$, and 150 mm, where l_d is calculated in accordance with 8.2.19 to develop f_y .
- 8.2.21.2 Where A_s provided is at least twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each reinforcement sheet shall not be less than the larger of $1.5 l_d$, and 50 mm, where l_d is calculated in accordance with 8.2.19 to develop f_y .

8.3 EARTHQUAKE-RESISTANT DESIGN PROVISIONS

8.3.1 Notation

- A_{ch} = cross-sectional area of a structural member measured out to out of transverse reinforcement, mm²
- A_{cp} = area of concrete section resisting shear of an individual pier or horizontal wall segment, mm²
- A_{cv} = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm²
- A_g = gross area of section, mm²
- A_j = effective cross-sectional area within a joint, see 8.3.7.3, in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of :
- Beam width plus the joint depth
 - twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side (See 8.3.7.3)

A_{sh}	=	total cross-sectional area of transverse reinforcement (including cross ties) within spacing s and perpendicular to dimension h_c
b	=	effective compressive flange width of a structural member, mm
b_w	=	web width or diameter of circular section, mm
d_b	=	bar diameter, mm
E	=	load effects of earthquake or related internal moments and forces
f'_c	=	specified compressive strength of concrete
f_y	=	specified yield strength of reinforcement
f_{yt}	=	specified yield strength of transverse reinforcement
h_c	=	cross-sectional dimension of column core measured centre to centre of confining reinforcement
h_w	=	height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered
l_d	=	development length for a straight bar
l_{dh}	=	development length for a bar with a standard hook
l_o	=	minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, mm
l_w	=	length of entire wall (diaphragm) or of segment of wall (diaphragm) considered in the direction of shear force
M_{pr}	=	probable flexural moment strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor ϕ of 1.0
M_s	=	portion of slab moment balanced by support moment
s	=	spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm
s_o	=	maximum spacing of transverse reinforcement, mm
V_c	=	nominal shear strength provided by concrete
V_e	=	design shear force
V_n	=	nominal shear strength
V_u	=	factored shear force at section
α_c	=	coefficient defining the relative contribution of concrete strength to wall strength
ρ	=	ratio of tension reinforcement = A_s/bd
ρ_g	=	ratio of total reinforcement area to cross-sectional area of column
ρ_n	=	ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv}
ρ_s	=	ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out to out)
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}
ϕ	=	strength reduction factor.

8.3.2 Definitions

BASE OF STRUCTURE : The level at which earthquake motions are assumed to be imparted to a structure. This level does not necessarily coincide with the ground level.

BOUNDARY MEMBERS : Members along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. These members do not necessarily require an increase in the thickness of the wall or diaphragm. If required, edges of openings within walls and diaphragms shall be provided with boundary members.

COLLECTOR ELEMENTS : Elements that are used to transmit the inertial forces within the diaphragms to members of the lateral force resisting systems.

CROSS TIE : A continuous bar having a hook not less than 135 deg with at least a six diameter extension at one end but not less than 75 mm, and a hook not less than 90 deg with at least a six diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90 deg hooks of two successive cross ties engaging the same longitudinal bars shall be alternated end for end.

DEVELOPMENT LENGTH FOR A BAR WITH A STANDARD HOOK : The shortest distance between the critical section and a tangent to the outer edge of the 90 deg hook.

HOOP : A hoop is a closed tie or continuously round tie. A closed tie can be made up of several reinforcing elements with 135 deg hooks having a six diameter extension at each end (but not less than 75 mm). A continuously round tie shall have at each end a 135 deg hook with a six diameter extension that engages the longitudinal reinforcement but not less than 75 mm.

LATERAL FORCE RESISTING SYSTEM : That portion of the structure composed of members designed to resist forces related to earthquake effects.

SHELL CONCRETE : Concrete outside the transverse reinforcement confining the concrete

STRUCTURAL DIAPHRAGMS : Structural members, such as floor and roof slabs, which transmit inertial forces to lateral force resisting members.

STRUCTURAL WALLS : Walls designed to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shear wall is a structural wall.

STRUT : An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

TIE ELEMENTS : Elements used to transmit inertial forces and prevent separation of building components.

8.3.3 General Requirements

8.3.3.1 Scope

- (a) This section 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.
- (b) The provisions of Chapter 6, shall apply except as modified by the provisions of this section.
- (c) Structures assigned to seismic design category SDC D (see Chapter 2), all reinforced concrete structures shall satisfy the requirements of special moment frames as given in 8.3.3 through 8.3.8 in addition to the requirements of Chapter 6.
- (d) Structures assigned to SDC C (see Chapter 2), all reinforced concrete structures shall be built to satisfy the requirements of intermediate moment frames as given in 8.3.10 in addition to the requirements of Chapter 6.
- (e) Structures assigned to SDC B (see Chapter 2), all reinforced concrete structures shall be built to satisfy the requirements of ordinary moment frames as given in 8.3.9 in addition to the requirements of Chapter 6.
- (f) Structures in lower SDCs are permitted to design with detailing provisions of higher SDCs to take advantage of lower design force levels.

8.3.3.2 Analysis and Proportioning of Structural Members

- (a) The interaction of all structural and nonstructural members shall be considered in the analysis.
- (b) Rigid members which are not a part of the lateral force resisting system are allowed 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.
- (c) Structural members below base of structure required to transmit forces resulting from earthquake effects to the foundation shall also comply with the requirements of this section.
- (d) All structural members which are not a part of the lateral force resisting system shall conform to 8.3.9.

8.3.3.3 Strength Reduction Factors

Strength reduction factors shall be in accordance with 6.2.3.2 to 6.2.3.4.

8.3.3.4 Concrete in Special Moment Frames and Special Structural Walls

Compressive strength f'_c of the concrete shall be not less than 21 N/mm^2 . Specified compressive strength of lightweight concrete, f'_c , shall not exceed 35 MPa unless demonstrated by experimental evidence. Modification factor λ for lightweight concrete in Sec 8.3 shall be in accordance with 6.1.8 unless noted otherwise.

8.3.3.5 Reinforcement in Special Moment Frames and Special Structural Walls

Reinforcement resisting earthquake induced flexural and axial forces in frames and wall boundary members shall comply with ASTM A706M, ASTM A615M and BDS ISO 6935-2: 2007(E). Reinforcement with $f_y = 275 \text{ N/mm}^2$ and $f_y = 420 \text{ N/mm}^2$ are allowed in these members if (a) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 125 N/mm^2 (retests shall not exceed this value by more than an additional 20 N/mm^2), and (b) the ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than 1.25. The value of f_{yt} used to compute the amount of confinement reinforcement shall not exceed 700 MPa . The value of f_y or f_{yt} used in the design of shear reinforcement shall conform to 6.4.3.2.

8.3.3.6 Welding

Reinforcement required by factored load combinations which include earthquake effect shall not be welded except as specified in 8.3.4.2(d) and 8.3.5.3(b). In addition, welding shall not be permitted on stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design.

8.3.4 Flexural Members of Special Moment Frames

8.3.4.1 Scope

Requirements of this section shall apply to special moment frame members; (i) resisting earthquake induced forces, and (ii) proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions:

- Factored axial compressive force on frame member shall not exceed $0.1A_g f'_c$.
- Clear span for the member, l_n shall not be less than four times its effective depth.
- The width to depth ratio shall be at least 0.3.
- The width shall not be (i) less than 250 mm and (ii) 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.

These requirements are shown in Fig. 8.3.1.

8.3.4.2 Longitudinal Reinforcement

- At any section of a flexural member and for the top as well as for the bottom reinforcement, the amount of reinforcement shall be not less than $1.4b_w d / f_y$ and the reinforcement ratio, ρ shall not exceed 0.025 (Fig. 8.3.2). At least two bars shall be provided continuously both top and bottom.
- The positive moment strength at the face of the joint shall be not less than one-half of the negative moment strength provided at that face as shown in Fig. 8.3.2. Neither the negative nor the positive moment strength at any section along the member length shall be less than one-fourth the maximum moment strength provided at the face of either joint.
- 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 $d/4$ nor 100 mm . Lap splices shall not be used (i) within the joints, (ii) within a distance of twice the member depth from the face of the joint, and (iii) at locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame. These requirements are shown in Fig. 8.3.3.
- Welded splices and mechanical connections conforming to 8.2.12.3(a) through 8.2.12.3(d) are allowed 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. Welded splices

and mechanical connections (Type 1) shall not be used within a distance equal to twice the member depth from the column or beam faces for special moment frames or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacement.

8.3.4.3 Transverse Reinforcement

- (a) Hoops shall be provided in the following regions of frame members:
 - i. At both ends of the flexural member, over a length equal to twice the member depth measured from the face of the supporting member toward midspan (Fig. 8.3.4).
 - ii. Over lengths equal to twice the member depth (Fig. 8.3.4), on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame.
- (b) The first hoop shall be located not more than 50 mm from the face of the supporting member (Fig. 8.3.4). Maximum spacing of the hoops shall not exceed (i) $d/4$, (ii) eight times the diameter of the smallest longitudinal bars, (iii) 24 times the diameter of the hoop bars, and (iv) 300 mm.
- (c) Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to 8.1.9.4(c), and where hoops are not required, stirrups with seismic hooks shall be spaced not more than $d/2$ throughout the length of the member (Fig. 8.3.4).
- (d) Hoops in flexural members are allowed to be made up of two pieces of reinforcement consisting of a U-stirrup having hooks not less than 135 deg with 6 diameter but not less than 75 mm extension anchored in the confined core and a cross tie to make a closed hoop (Fig. 8.3.5). Consecutive cross ties engaging the same longitudinal bar 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.

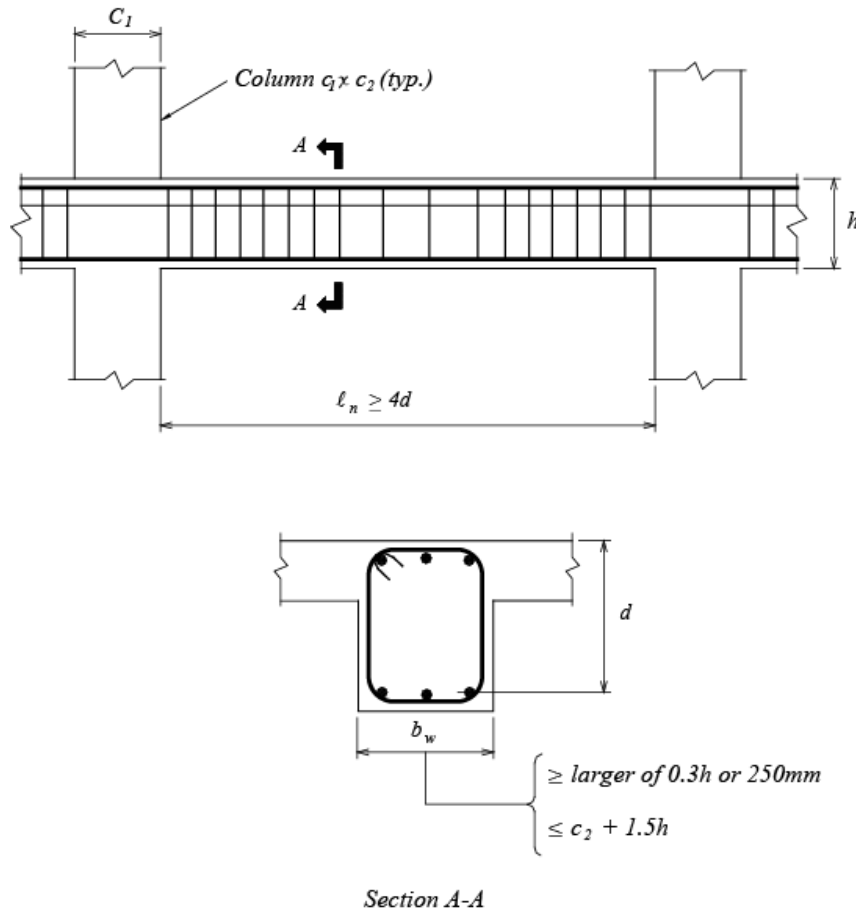


Fig. 8.3.1. General requirement for Flexural Members of Special Moment Frames

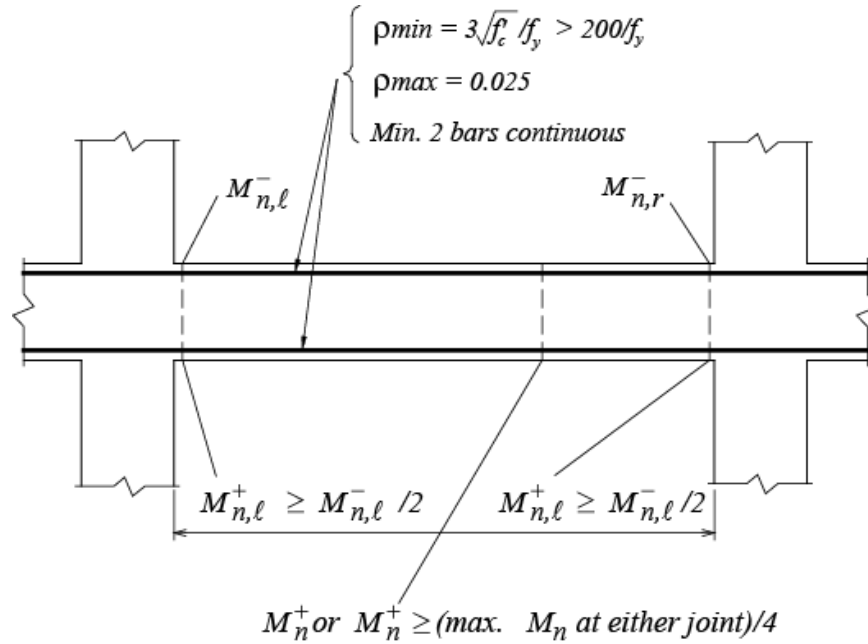
8.3.5 Special Moment Frame Members Subjected to Bending and Axial Load

8.3.5.1 Scope

The requirements of this section shall apply to columns and other frame members serving to resist earthquake forces and having a factored axial force exceeding $0.1A_g f'_c$. These frame members shall also satisfy the following conditions:

- The shortest cross-sectional dimension shall not be less than 300 mm.
- The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

These requirements are shown in Fig. 8.3.6.



Note: transverse reinforcement not shown for clarity

Fig. 8.3.2 Flexural Requirements for Flexural Members of Special Moment Frames

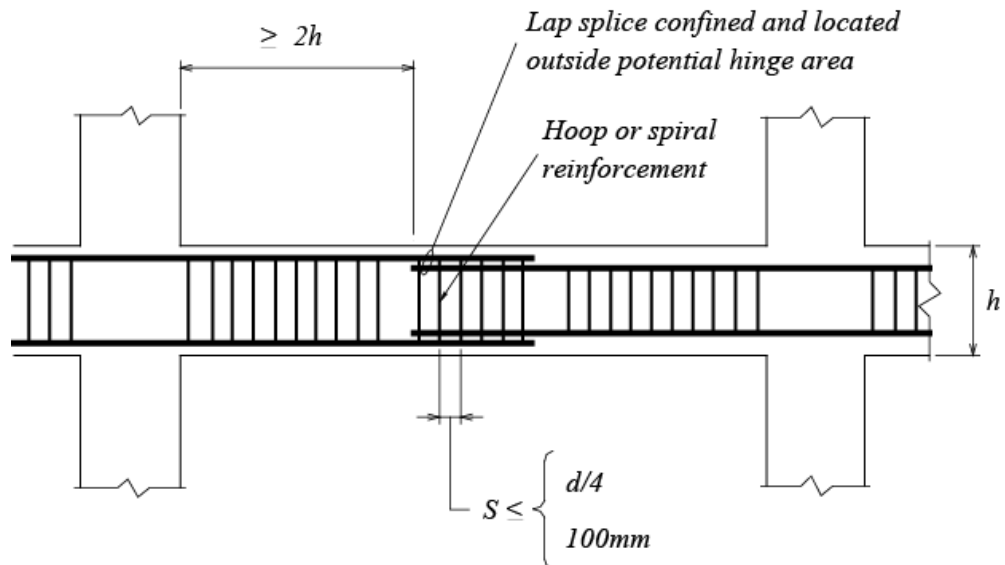


Fig. 8.3.3 Lap Splice Requirements for Flexural Members of Special Moment Frames

8.3.5.2 Minimum Flexural Strength of Columns

- (a) Flexural strength of any column designed to resist a factored axial compressive force exceeding $0.1A_g f'_c$ shall satisfy (b) or (c) below. Lateral strength and stiffness of columns not satisfying (b) below shall be ignored in calculating the strength and stiffness of the structure but shall conform to 8.3.9.
- (b) The flexural strength of the columns shall satisfy the following relation:

$$\sum M_c \geq 1.2 \sum M_g \quad (8.3.1)$$

where

$\sum M_c$ = sum of moments, at the centre of the joint, corresponding to the design flexural strength of the columns framing into that joint. The lowest flexural strength of the columns, calculated for the factored axial force, consistent with the direction of the lateral forces considered, shall be used.

$\sum M_g$ = 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 (8.3.1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

- (c) If the requirements of (b) above is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in 8.3.5.4 over their entire height.

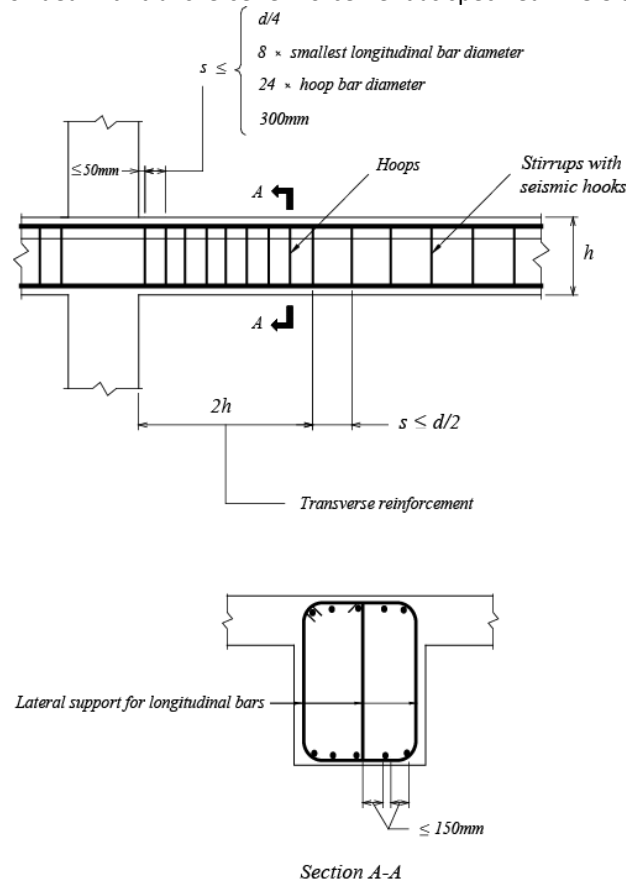


Fig. 8.3.4 Transverse Reinforcement Requirements for Flexural Members of Special Moment Frames

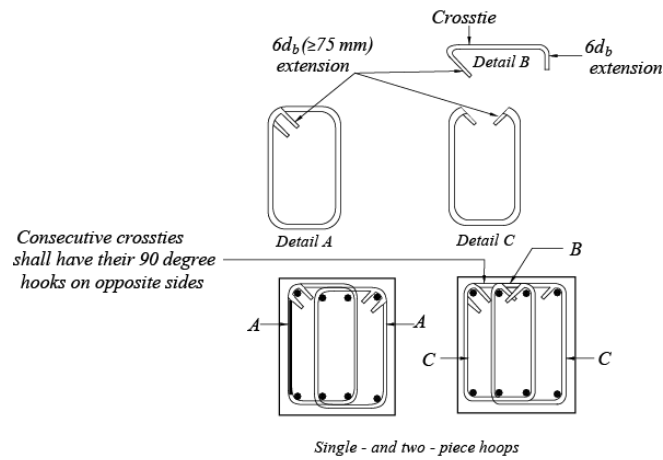


Fig. 8.3.5 Hoop Reinforcement Requirements for Flexural Members of Special Moment Frames

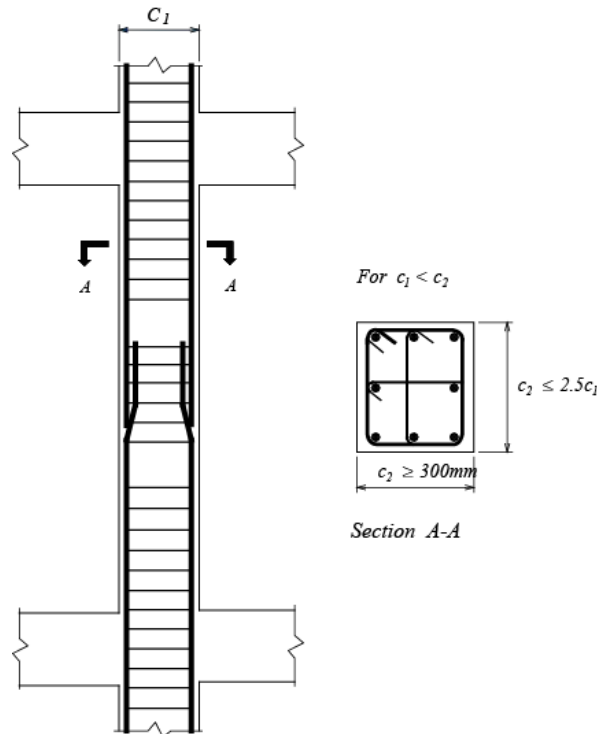


Fig. 8.3.6 General Requirements for frames subjected to bending and axial load

8.3.5.3 Longitudinal Reinforcement

- The reinforcement ratio, ρ_g , shall not be less than 0.01 and shall not exceed 0.06.
- Lap splices are permitted only within the centre half of the member length and shall be designed as tension splices. Welded splices and mechanical connections conforming to 8.2.12.3(a) through 8.2.12.3(d) are allowed 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.

These provisions are shown in Fig. 8.3.7.

8.3.5.4 Transverse Reinforcement

(a) Transverse reinforcement shall be provided as specified below and shown in Fig. 8.3.8 and Fig. 8.3.9 unless a larger amount is required by 8.3.8.

- i) The volumetric ratio of spiral or circular hoop reinforcement, ρ_s shall not be less than that indicated by the following equation:

$$\rho_s = \frac{0.12f'_c}{f_{yt}} \quad (8.3.2)$$

and shall not be less than that required by Eq (6.3.6).

- ii) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by the following equations:

$$A_{sh} = 0.3(sh_c f'_c / f_{yt})[(A_g / A_{ch}) - 1] \quad (8.3.3)$$

$$A_{sh} = \frac{0.09sh_c f'_c}{f_{yt}} \quad (8.3.4)$$

- iii) Transverse reinforcement shall be provided by either single or overlapping hoops or cross ties of the same bar size and spacing. 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.
- iv) If the design strength of member core satisfies the requirements of the specified loading combinations including earthquake effect, Eq (8.3.3) and (6.3.6) need not be satisfied.

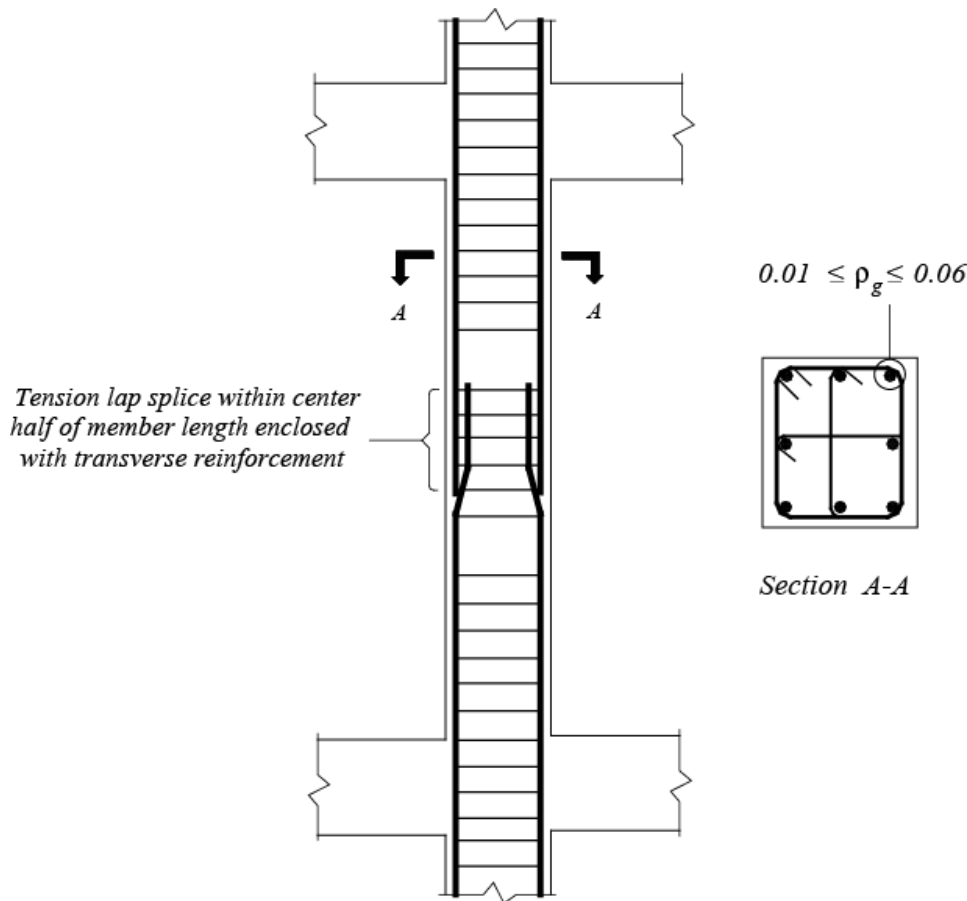


Fig. 8.3.7 Longitudinal Reinforcement Requirements

- (b) Transverse reinforcement shall not be spaced more than one-quarter of the minimum member dimension nor 100 mm.
- (c) Spacing of cross ties or legs of overlapping hoops shall not be more than 350 mm on centre in the direction perpendicular to the longitudinal axis of the member.

- (d) The volume of transverse reinforcement in amount specified in (a) through (c) above shall be provided over a length l_o from each joint face and on both sides of any section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame. The length l_o shall not be less than (i) the depth of the member at the joint face or at the section where flexural yielding is likely to occur, (ii) one-sixth of the clear span of the member, and (iii) 450 mm.
- (e) If the factored axial force in columns supporting reactions from discontinued stiff members, such as walls, exceeds $0.1A_g f'_c$ they shall be provided with transverse reinforcement as specified in (a) through (c) above over their full height beneath the level at which the discontinuity occurs. Transverse reinforcement shall extend into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column in accordance with 8.3.7.4. If the lower end of the column terminates on a wall, transverse reinforcement as specified above shall extend into the wall for at least the development length of the largest longitudinal reinforcement in the column at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as specified in above shall extend at least 300 mm into the footing or mat.
- (f) Where transverse reinforcement as specified in (a) through (c) above, is not provided throughout the full length of the column, the remainder of the column length shall contain spiral or hoop reinforcement with centre to centre spacing not exceeding the smaller of 6 times the diameter of the longitudinal column bars or 150 mm.

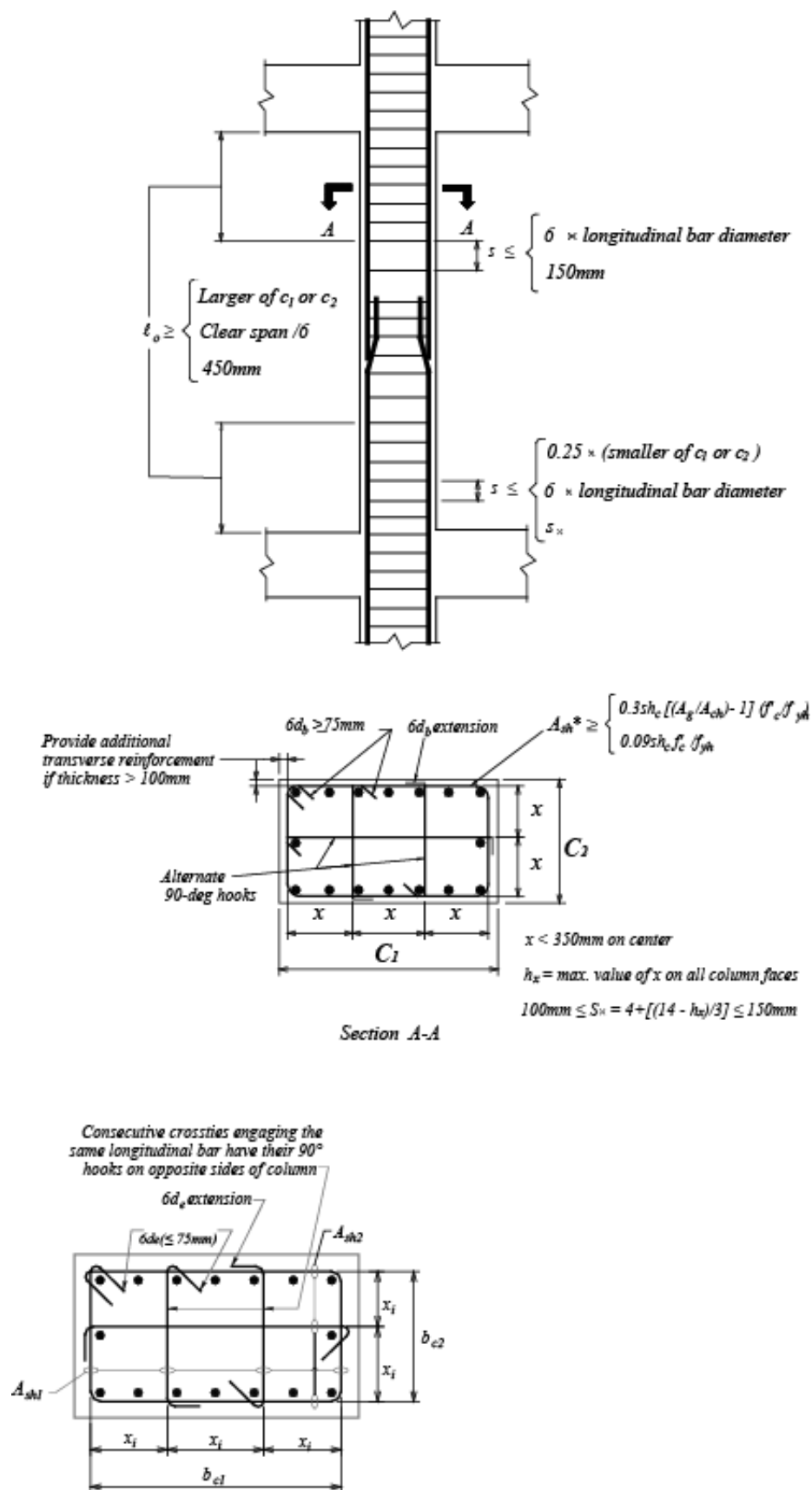


Fig. 8.3.8 Transverse Reinforcement Requirements- Rectangular Hoop

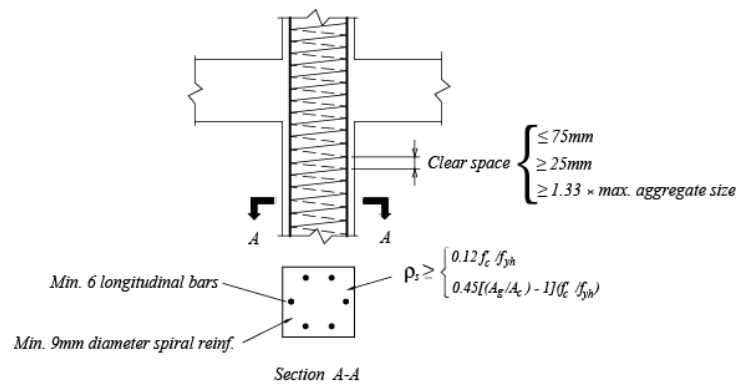


Fig. 8.3.9 Transverse Reinforcement Requirements- Spiral Hoop

8.3.6 Special Structural Walls and Diaphragms

8.3.6.1 Scope

The requirements of this section apply to structural walls 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.

8.3.6.2 Reinforcement

- The reinforcement ratio, ρ_v for structural walls shall not be less than 0.0025 along the longitudinal and transverse directions. Reinforcement spacing each way shall not exceed 450 mm. Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane. If the design shear force does not exceed $0.083A_{cv}\lambda\sqrt{f'_c}$, the shear reinforcement may conform to 6.6.3.
- At least two layers of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $0.17A_{cv}\lambda\sqrt{f'_c}$.
- Structural truss members, struts, ties, and collector members with compressive stresses exceeding $0.2 f'_c$ shall have special transverse reinforcement, as specified in 8.3.5.4 over the total length of the member. The special transverse reinforcement is allowed to be discontinued at a section where the calculated compressive stress is less than $0.15 f'_c$. Stresses shall be calculated for the factored forces using a linear elastic model and gross section properties of the members considered.
 - 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 8.3.7.4.

8.3.6.3 Boundary Members for Structural Walls and Diaphragms

- Boundary members shall be provided at boundaries and edges around openings of structural walls and diaphragms for which the maximum extreme fibre stress exceeds $0.2 f'_c$ unless the entire wall or diaphragm member is reinforced to satisfy 8.3.5.4(a) through 8.3.5.4(c). The boundary members may be discontinued where the calculated compressive stress is less than $0.15 f'_c$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties.
- Where required, boundary members shall have transverse reinforcement as specified in 8.3.5.4(a) through 8.3.5.4(c).
- Boundary members of structural walls shall be designed 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.
- Boundary members of structural diaphragms shall be proportioned to resist the sum of the factored axial 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.
- Transverse reinforcement in walls with boundary members shall be anchored within the confined core of the boundary member to develop the tensile yield stress.
- Transverse reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be

enclosed in the U-stirrups having the same size and spacing as, and spliced to, the transverse reinforcement, except when V_u/V_u in the plane of the wall is less than $0.083A_{cv}\lambda\sqrt{f'_c}$.

8.3.6.4 Construction Joints

All construction joints in walls and diaphragms shall conform to 5.16.4 and contact surfaces shall be roughened as specified in 6.4.5.9.

8.3.6.5 Discontinuous Walls

Columns supporting discontinuous walls shall be reinforced in accordance with 8.3.5.4(e).

8.3.7 Joints of Special Moment Frames

8.3.7.1 General Requirements

- Forces in longitudinal beam reinforcement at the faces of joints of reinforced concrete frames shall be determined for a stress of $1.25 f_y$ in the reinforcement.
- Joint strength shall be calculated by the appropriate strength reduction factors specified in 6.2.3.1.
- 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 8.3.7.4 below and in compression according to Sec 8.2.

8.3.7.2 Transverse Reinforcement

- As specified in 8.3.5.4, transverse hoop reinforcement shall be provided within the joint, unless the joint is confined by structural members as specified in (b) below.
- Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by 8.3.5.4(a) shall be provided where members frame into all four sides of the joint and where each member width is at least three-fourths the column width. At these locations, the spacing specified in 8.3.5.4(b) may be increased to 150 mm.
- As required by 8.3.5.4, transverse reinforcement 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.

These provisions are shown in Fig.8.3.10 and Fig.8.3.11.

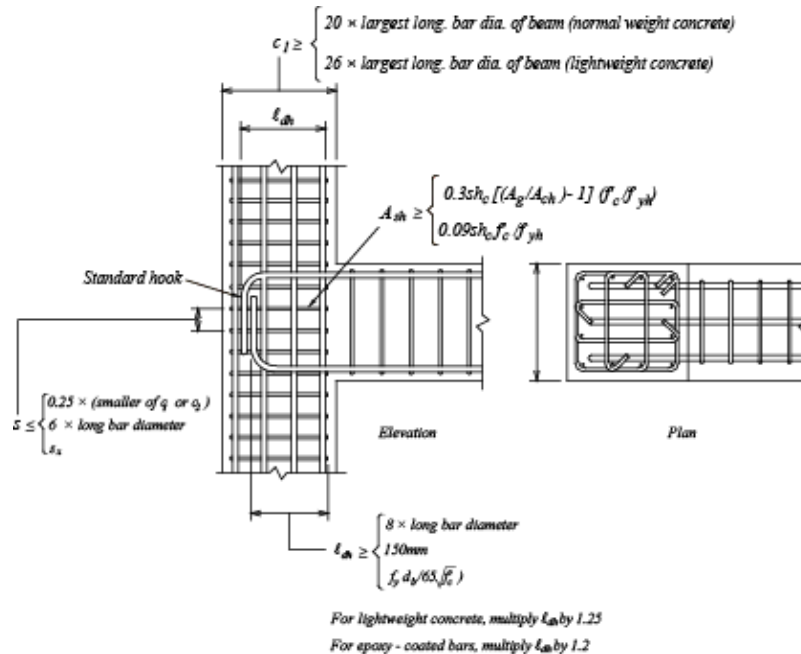


Fig. 8.3.10 General Requirements and Transverse Reinforcement Requirements for Joints not confined by Structural member

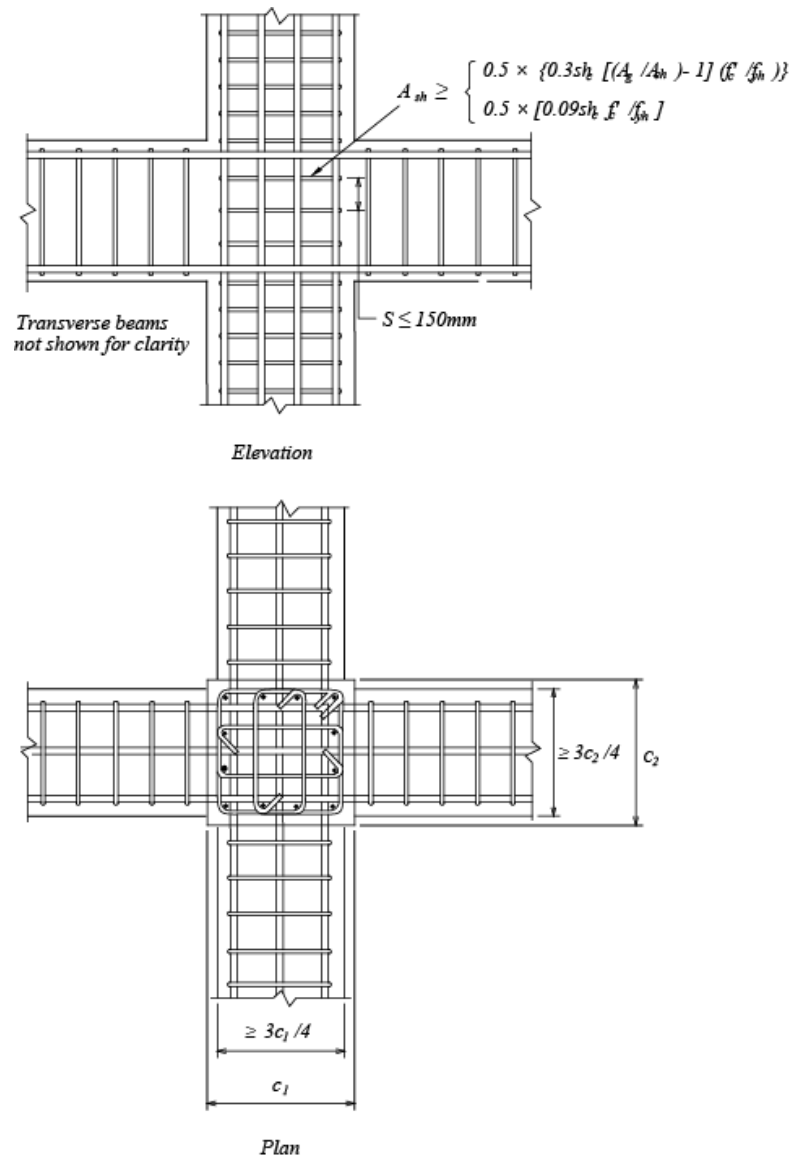


Fig. 8.3.11 Transverse Reinforcement Requirements for Joints Confined by Structural member

8.3.7.3 Shear Strength

The nominal shear strength for the joint shall be taken not greater than the forces specified below :

$1.7\sqrt{f'_c}A_j$ for joints confined on all four faces

$1.2\sqrt{f'_c}A_j$ for joints confined on three faces or on two opposite faces

$1.0\sqrt{f'_c}A_j$ for others

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

8.3.7.4 Development Length of Bars in Tension

- (a) The development length, l_{dh} , for a bar with a standard 90° hook shall be not less than (i) $8d_b$, (ii) 150 mm, and (iii) the length required by Eq (8.3.5).

$$l_{dh} = \frac{f_y d_b}{5.4 \sqrt{f'_c}} \quad (8.3.5)$$

for bar sizes 10 mm ϕ through 36 mm ϕ .

- (b) For bar sizes 10 mm ϕ through 36 mm ϕ , the development length, l_d , for a straight bar shall be not less than (i) 2.5 times the length required by (a) above, if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm, and (ii) 3.5 times the length required by (a) above, if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.
- (c) 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.

8.3.8 Shear Strength Requirements

8.3.8.1 Design Forces

- (a) 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 M_{pr} act at the joint faces, and that the member is loaded with the factored tributary gravity load along its span.
- (b) Frame Members Subjected to Combined Bending and Axial Load: The design shear force V_e shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths M_{pr} of the member associated with the range of factored axial loads on the member. The member shears need not exceed those determined from joint strengths based on the probable moment strength M_{pr} of the transverse members framing into the joint. In no case, V_e shall be less than the factored shear determined by the analysis of the structure.
- (c) Structural Walls and Diaphragms: The design shear force V_e shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in Chapter 2, loads.

8.3.8.2 Transverse Reinforcement in Frame Members

- (a) For determining the required transverse reinforcement in frame members, the quantity V_c shall be assumed to be zero if the factored axial compressive force including earthquake effects is less than $0.05A_gf'_c$ when the earthquake-induced shear forces, calculated in accordance with 8.3.8.1(a), represents one-half or more of total design shear.
- (b) Stirrups or ties required to resist shear shall be closed hoops over lengths of members as specified in 8.3.4.3, 8.3.5.4 and 8.3.7.2.

8.3.8.3 Shear Strength of Special Structural Walls and Diaphragms

- (a) Nominal shear strength of structural walls and diaphragms shall be determined using either (b) or (c) below.
- (b) Nominal shear strength, V_n of structural walls and diaphragms shall be assumed not to exceed the shear force calculated from

$$V_n = A_{cv}(0.17\lambda\sqrt{f'_c} + \rho_n f_y) \quad (8.3.6)$$

- (c) For walls and wall segments having a ratio of (h_w/l_w) less than 2.0, nominal shear strength of wall and diaphragm shall be determined from

$$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_n f_y) \quad (8.3.7)$$

where the coefficient α_c varies linearly from 0.25 for $(h_w/l_w) = 1.5$ to 0.17 for $(h_w/l_w) = 2.0$. For $(h_w/l_w) \leq 1.5$, $\alpha_c = 0.25$.

- (d) Value of ratio (h_w/l_w) used in (c) above 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.
- (e) Walls and diaphragms shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. 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 .
- (f) Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed $0.67A_{cv}\sqrt{f'_c}$, 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 $0.83A_{cp}\sqrt{f'_c}$ where A_{cp} represents the cross-sectional area of the pier considered.

- (g) Nominal shear strength of horizontal wall segments shall be assumed not to exceed $0.83A_{cp}\sqrt{f'_c}$, where A_{cp} represents the cross-sectional area of a horizontal wall segment.

8.3.9 Ordinary Moment Frame Members not Proportioned to Resist Forces Induced by Earthquake Motion

8.3.9.1 Induced moments

Frame members assumed not to contribute to lateral resistance shall be detailed according to (a) or (b) below depending on the magnitude of moments induced in those members when subjected to twice the lateral displacement under the factored lateral forces.

- (a) Members with factored gravity axial forces not exceeding $0.1A_gf'_c$ shall satisfy 8.3.4.2(a) and 8.3.8.1(a) and members with factored gravity axial forces exceeding $0.1A_gf'_c$ shall satisfy 8.3.5.4, 8.3.7.2(a) and 8.3.8.1(b) when the induced moment exceeds the design moment strength of the frame member.
- (b) The member shall satisfy 8.3.4.2(a) when the induced moment does not exceed the design moment strength of the frame members.

8.3.9.2 Tie requirements

All frame members with factored axial compressive forces exceeding $0.1A_gf'_c$ shall satisfy the following special requirements unless they comply with 8.3.5.4.

- (a) Ties shall have hooks not less than 135° with extensions not less than 6 tie bar diameter or 60 mm. Cross ties as defined in 8.3.2 are allowed.
- (b) 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 (i) eight diameters of the smallest longitudinal bar enclosed, (ii) 24 tie bar diameters, and (iii) one-half the least cross-sectional dimension of the column. The length l_o shall not be less than (i) one-sixth of the clear height of the column, (ii) the maximum cross-sectional dimension of the column, and (iii) 450 mm.
- (c) The first tie shall be within a distance equal to $0.5s_o$ from the face of the joint.
- (d) The tie spacing shall not exceed $2s_o$ in any part of the column.

8.3.10 Requirements for Intermediate Moment Frames

8.3.10.1 Scope

For structures assigned to SDC C, structural frames proportioned to resist forces induced by earthquake motions shall satisfy the requirements of 8.3.10 in addition to those of Chapter 6.

8.3.10.2 Reinforcement Requirements

Reinforcement details in a frame member shall satisfy 8.3.10.4 below if the factored compressive axial load for the member does not exceed $0.1A_gf'_c$. If the factored compressive axial load is larger, frame reinforcement details shall satisfy 8.3.10.5 below unless the member has spiral reinforcement according to Eq (6.3.6). 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 8.3.10.6 below.

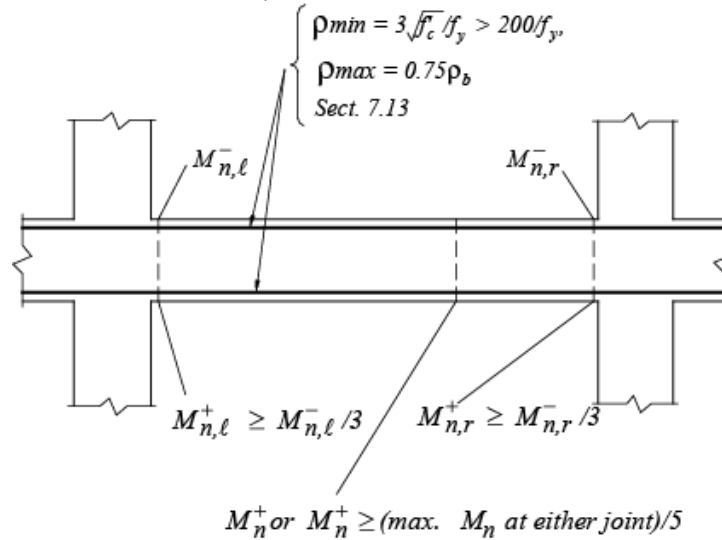
8.3.10.3 Shear requirements

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.

8.3.10.4 Beams

- (a) The positive moment strength at the face of the joint shall not be less than one-third the negative moment strength provided at that face (Fig.8.3.12). Neither the negative nor positive moment strength at any section along the length of the member shall be less than one-fifth of the maximum moment strength provided at the face of either joint.

- (b) 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 midspan (Fig.8.3.13). The first stirrup shall be located not more than 50 mm from the face of the supporting member. Maximum stirrup spacing shall not exceed (a) $d/4$, (b) 8 times the diameter of the smallest longitudinal bar enclosed, (c) 24 times the diameter of the stirrup bar, and (d) 300 mm.
- (c) Stirrups shall be placed at not more than $d/2$ throughout the length of the member.



Note: transverse reinforcement not shown for clarity

Fig.8.3.12 Flexural Requirements for Beams

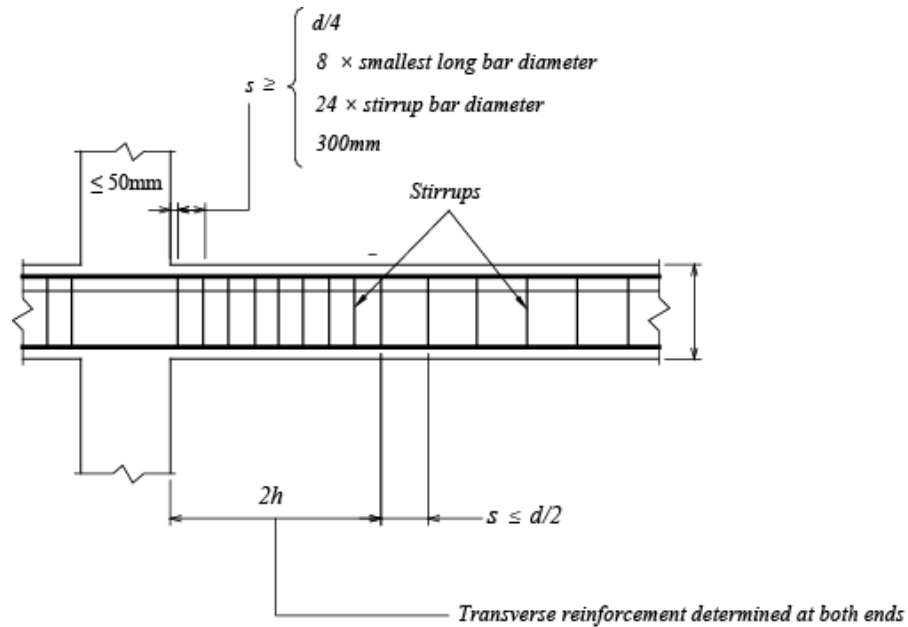


Fig.8.3.13 Transverse reinforcement Requirements for Beams

8.3.10.5 Columns

- (a) Maximum tie spacing shall not exceed s_o so over a length l_o measured from the joint face. The spacing s_o so shall not exceed (i) 8 times the diameter of the smallest longitudinal bar enclosed, (ii) 24 times the diameter of the tie bar, (iii) one-half of the smallest cross-sectional dimension of the frame member, and (iv) 300 mm. The length l_o shall not be less than (i) one-sixth of the clear span of the member, (ii) maximum cross-sectional dimension of the member, and (iii) 450 mm.
- (b) The first tie shall be located not more than $s_o/2$ from the joint face.
- (c) Joint reinforcement shall conform to 6.4.9.

(d) Tie spacing shall not exceed $2s_o$ throughout the length of the member.

These requirements are shown in Fig. 8.3.14.

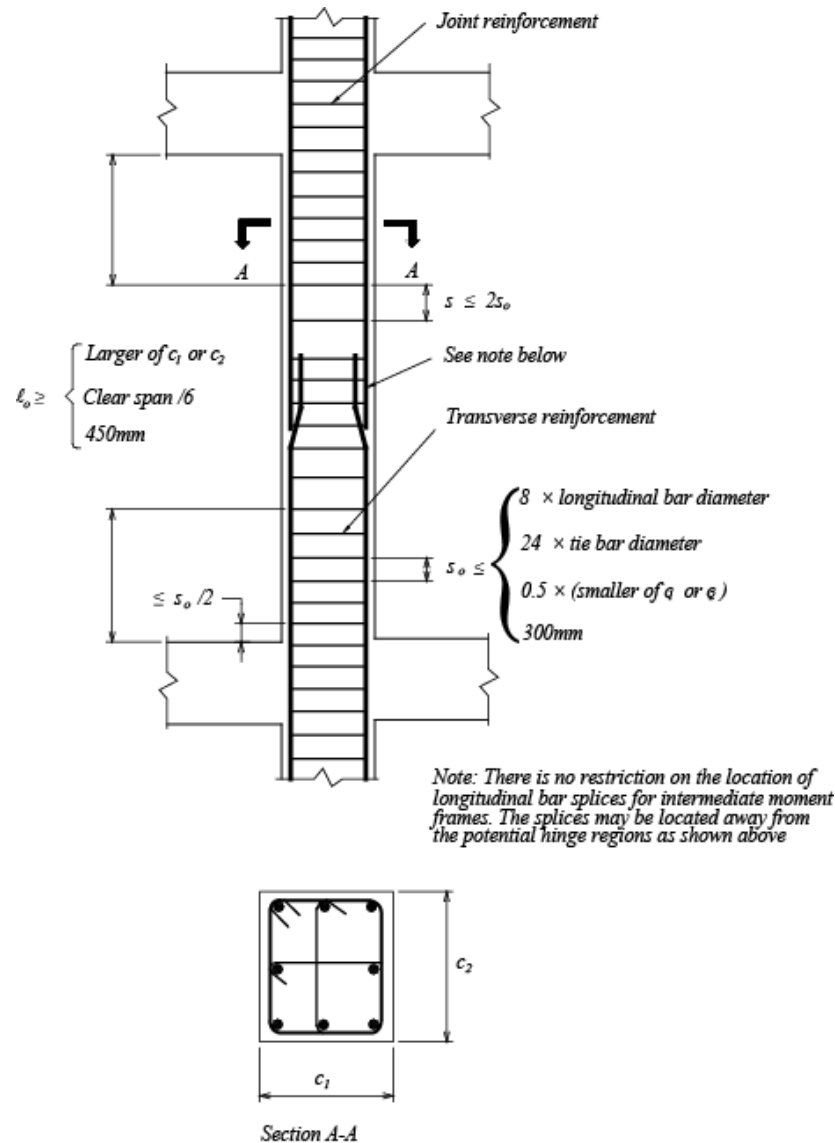
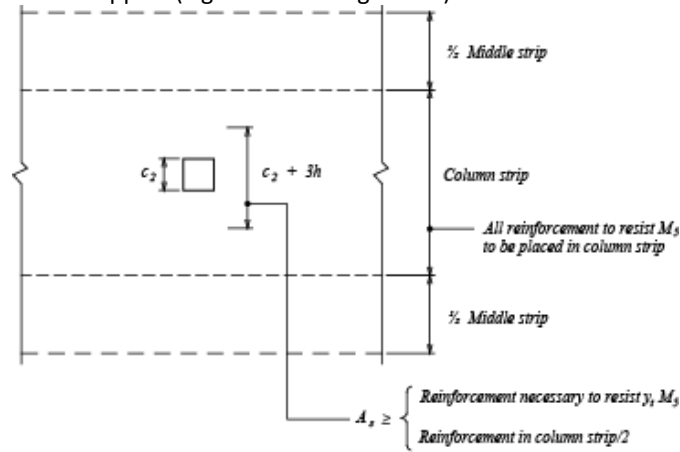


Fig.8.3.14 Transverse Reinforcement Requirements for Columns

8.3.10.6 Two-way Slabs without Beams

- The factored slab moment at the supports relating to earthquake effect shall be determined for load combinations specified in Chapter 2, Loads. All reinforcement provided to resist the portion of slab moment balanced by support moment shall be placed within the column strip defined in 6.5.2.1 (Fig.8.3.15).
- The fractional part of the column strip moment shall be resisted by reinforcement placed within the effective width (Fig.8.3.15) specified in 6.5.5.3.2.
- Not less than one-half of the total reinforcement in the column strip at the support shall be placed within the effective slab width (Fig.8.3.15) specified in 6.5.5.3.2.
- Not less than one-quarter of the top steel at the support in the column strip shall be continuous throughout the span (Fig.8.3.16).
- Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.
- Not less than one-half of all bottom reinforcement at midspan shall be continuous and shall develop its yield strength at the face of support (Fig.8.3.17).

- (g) At discontinuous edges of the slab all top and bottom reinforcement at the support shall be developed at the face of the support (Fig.8.3.16 and Fig.8.3.17).



Note: applies to both top and bottom reinforcement

Fig.8.3.15 Reinforcement Details at Support of Two-way Slabs without beams

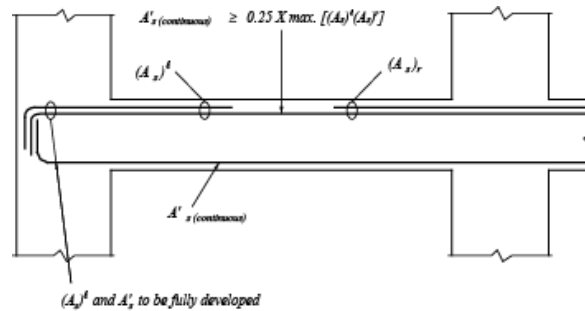


Fig.8.3.16 Reinforcement Details in Two-way Slabs without beams: Column Strip

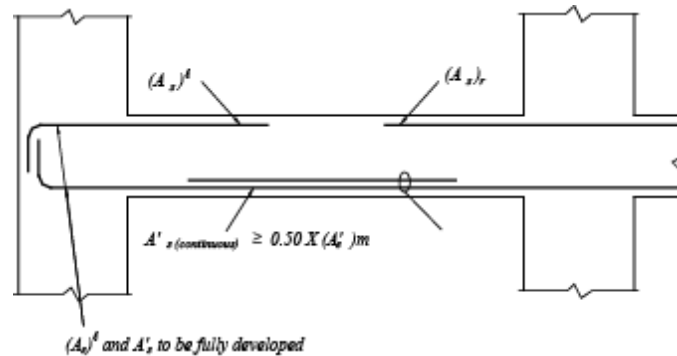


Fig.8.3.17 Reinforcement Details in Two-way Slabs without beams: Middle Strip

PRESTRESSED CONCRETE STRUCTURES

Part A - Design

9.1 Scope

- 9.1.1** Provisions of this chapter shall apply to members prestressed with wires, strands, or bars conforming to the specifications of prestressing tendons given in Article 9.5.1.3.
- 9.1.2** All provisions of this code not specifically excluded, and not in conflict with provisions of this Chapter 9, shall apply to prestressed concrete.

9.2 Definitions

ACTION: Mechanical force or environmental effect to which the structure (or structural component) is subjected.

AERODYNAMIC SHAPE FACTOR: Factor to account for the effect of geometry of structure on the surface pressure due to wind.

ANALYSIS (ASSESSMENT): Acceptable methods of evaluating the performance indices or verifying the compliance of specific criteria.

ANCHORAGE: In post-tensioning, a mechanical device used to anchor the tendon to the concrete; in pretensioning, a device used to anchor the tendon until the concrete has reached a pre-determined strength, and the prestressing force has been transferred to the concrete; for reinforcing bars, a length of reinforcement, or a mechanical anchor or hook, or combination thereof at the end of a bar needed to transfer the force carried by the bar into the concrete.

ANCHORAGE BLISTER: A build-up area on the web, flange, or flange-web junction for the incorporation of tendon anchorage fittings.

ANCHORAGE ZONE: The portion of the structure in which the prestressing force is transferred from the anchorage device on to the local zone of the concrete, and then distributed more widely in the general zone of the structure.

AT JACKING: At the time of tensioning the prestressing tendons.

AT LOADING: The maturity of the concrete when loads are applied. Such loads include prestressing forces and permanent loads but generally not live loads.

AT TRANSFER: Immediately after the transfer of prestressing force to the concrete.

AUTOGENEOUS SHRINKAGE: Volume decrease due to loss of water in the hydration process causing negative pore pressure in concrete.

BIOLOGICAL DEGRADATION: The physical or chemical degradation of concrete due to the effect of organic matters such as bacteria, lichens, fungi, moss, etc.

BLEEDING: Segregation between water and the other ingredients in concrete causing water to rise up to the surface of the freshly placed concrete.

BONDED MEMBER: A prestressed concrete member in which tendons are bonded to the concrete either directly or through grouting.

BONDED POST-TENSIONING: Post-tensioned construction in which the annular space around the tendons is grouted after stressing, thereby bonding the tendon to the concrete section.

BONDED TENDON: Prestressing tendon that is bonded to concrete either directly or through grouting.

BURSTING FORCE: Tensile forces in the concrete in the vicinity of the transfer or anchorage of prestressing forces.

CARBONATION: Action caused by chemical reaction between calcium hydroxide in concrete and carbon dioxide in the environment, resulting in a denser surface for the carbonated concrete and reduction of alkalinity in the carbonated portion.

CAST-IN-PLACE CONCRETE: Concrete placed in its final position in the structure while still in a plastic state.

CHARACTERISTIC STRENGTH: Unless otherwise stated in this code, the characteristic strength of material refers to the value of the strength below which none of the test results should fall below by more than 15% or 3.5 MPa for ≤ 35 MPa concrete, and 10% or 3.5 MPa for ≥ 35 MPa concrete, whichever is larger.

CHEMICAL ADMIXTURES: Admixtures which are usually used in small quantities typically in the form of liquid and can be added to the concrete both at the time of mixing and before placing to improve various concrete properties such as workability, air content and durability, etc.

CLOSELY SPACED ANCHORAGES: Anchorage devices are defined as closely spaced if their centre to centre spacing does not exceed 1.5 times the width of the anchorage devices in the direction considered.

CLOSURE: A placement of cast-in-place concrete used to connect two or more previously cast portions of a structure.

COMPOSITE CONSTRUCTION: Concrete components or concrete and steel components interconnected to respond to force effects as a unit.

COMPRESSION-CONTROLLED SECTION: A cross-section in which the net tensile strain in the extreme tension steel at nominal resistance is less than or equal to the compression-controlled strain limit.

COMPRESSION-CONTROLLED STRAIN LIMIT: The net tensile strain in the extreme tension steel at balanced strain conditions.

CONCRETE COVER: The specified minimum distance between the surface of the reinforcing bars, strands, post-tensioning ducts, anchorages, or other embedded items, and the surface of the concrete.

CONFINEMENT: A condition where the disintegration of the concrete under compression is prevented by the development of lateral and/or circumferential forces such as may be provided by appropriate reinforcing steel or composite tubes, or similar devices.

CONFINEMENT ANCHORAGE: Anchorage for a post-tensioning tendon that functions on the basis of containment of the concrete in the anchorage zone by special reinforcement.

CREEP: Time dependent deformation of concrete under permanent load.

CREEP COEFFICIENT: The ratio of creep strain to elastic strain in concrete.

CREEP IN CONCRETE: Increase in strain with time in concrete subjected to sustained stress.

CURVATURE FRICTION: Friction resulting from bends or curves in the specified prestressing tend stage at which the compressive stresses on profile.

DAMAGE CONTROL: A means to ensure that the limit state requirement is met for restorability or repairability of a structure.

DECOMPRESSION: The stage at which the compressive stresses, induced prestress, are overcome by the tensile stresses.

DEEP COMPONENT: components in which the distance from the point of 0.0 shear to the face of the support is less than $2d$ or components in which a load causing more than one-third of the shear at a support is closer than $2d$ from the face of the support.

DEFORMABILITY: A term expressing the ability of concrete to deform.

DEGREE OF DETERIORATION: The extent to which the performance of a structure is degraded or the extent to which the deterioration has progressed from the time of construction, as a result of its exposure to the environment.

DESIGN LIFE: Assumed period for which the structure is to be used satisfactorily for its intended purpose or function with anticipated maintenance but without substantial repair being necessary.

DETERIORATION FACTOR: The factor affecting the deterioration process.

DETERIORATION INDEX: An index selected for estimating and evaluating the extent of the deterioration process.

DETERIORATION PREDICTION: Prediction of the future rate of deterioration of a structure based on results of inspection and relevant records made during the design and construction stages.

DEVIATION SADDLE: A concrete block build-out in web, flange, or web-flange junction used to control the geometry of or to provide a means for changing direction of, external tendons.

DRYING SHRINKAGE: Volume decrease due to loss of moisture from concrete in the hardened state which is usually serious in hot and dry environment.

DURABILITY DESIGN: Design to ensure that the structure can maintain its required functions during service life under environmental actions.

DURABILITY GRADE: The extent of durability to which the structure shall be maintained in order to satisfy the required performance during its design life. This affects the degree and frequency of the remedial actions to be carried out during that life.

DURABILITY LIMIT STATE: The maximum degree of deterioration allowed for the structure during its design life.

DURABILITY PREDICTION: Prediction of the future degree of deterioration of the structure based on data used in its design.

DYNAMIC APPROACH: An approach based on dynamic analysis to assess the overall forces on a structure liable to have a resonant response to wind action.

DYNAMIC RESPONSE FACTOR: Factor to account for the effects of correlation and resonant response.

EARLY AGE STATE: The state of concrete from final setting until the achievement of the required characteristic strength.

EFFECTIVE PRESTRESS : Stress remaining in prestressing tendons after all losses have occurred, excluding effects of dead load and superimposed load.

ENVIRONMENTAL ACTIONS: An assembly of physical, chemical or biological influences which may cause deterioration to the materials making up the structure, which in turn may adversely affect its serviceability, restorability and safety.

FATIGUE LOADS: Repetitive loads causing fatigue in the material which reduces its strength, stiffness and deformability.

FINAL PRESTRESS : Stress which exists after substantially all losses have occurred.

FINAL TENSION : The tension in the steel corresponding to the state of the final prestress.

FORMWORK: Total system of support for freshly placed concrete including the mould or sheathing, all supporting members, hardware and necessary bracings.

FRESH STATE OF CONCRETE: The state of concrete after mixing until the completion of placing.

FUNCTION: The task which a structure is required to perform.

GENERAL ZONE: Region adjacent to a post-tensioned anchorage within which the prestressing force spreads out to an essentially linear stress distribution over the cross section of the component.

GROUT: A mixture of cementitious material and water with or without admixtures.

HARDENED STATE OF CONCRETE: The state of concrete after achieving the required strength.

IMPORTANCE: rank assigned to a structure according to the likely overall impact caused by its failure, due to deterioration, to satisfactorily perform its functions as determined at the time of design.

INITIAL PRESTRESS : The prestress in the concrete at transfer.

INITIAL TENSION : The maximum stress induced in the prestressing tendon at the time of stressing operation.

IRREGULAR STRUCTURES: Structures having unusual shapes such as open structures, structures with large overhangs or other projections, and any building with a complex shape.

JACKING FORCE : Temporary force exerted by device that introduces tension into prestressing tendons.

LIMIT STATE: A critical state specified using a performance index, beyond which the structure no longer satisfies the design performance requirements.

LIMITS OF DISPLACEMENT: Allowable deformation of structure in terms of such parameters as inter-storey drift and relative horizontal displacement, to control excessive deflection, cracking and vibration.

Long-term performance index: Index defining the remaining capacity of a structure in performing its design functions during the design life.

LOCAL ZONE: The volume of concrete that surrounds and is immediately ahead of the anchorage device and that is subjected to high compressive stresses.

MAINTENANCE: A set of activities taken to ensure that the structure continues to perform its functions satisfactorily during the design life.

MECHANICAL FORCES: An assembly of concentrated or distributed forces acting on a structure, or deformations imposed on it.

MODEL: Mathematical description or experimental setup simulating the actions, material properties and behavior of a structure.

MONITORING: Continuous recording of data pertaining to deterioration and/or performance of structure using appropriate equipment.

NOMINAL STRENGTH OF MATERIAL: The characteristic values of the strength of materials used for calculation, in absence of the available statistical data.

NORMAL CONCRETE: Concrete which is commonly used in construction; it does not include special constituent materials other than Portland cement, water, fine aggregate, coarse aggregate and common mineral and chemical admixtures; it does not require any special practice for its manufacturing and handling.

OVERALL PERFORMANCE INDEX: Index indicating the overall performance of the structure.

special practice for its manufacturing and handling.

PARTIAL PERFORMANCE INDEX: Index indicating a partial performance of the structure.

PARTIAL SAFETY FACTOR FOR MATERIAL: For analysis purposes, the design strength of a material is determined as the characteristic strength divided by a partial safety factor.

PERFORMANCE: Ability (or efficiency) of a structure to perform its design functions.

PERFORMANCE INDEX: Index indicating structural performance quantitatively.

PERMANENT ACTIONS: Self-weights of structures inclusive of permanent attachments, fixtures and fittings.

PLASTIC SHRINKAGE: Shrinkage arising from loss of water from the exposed surface of concrete during the plastic state, leading to cracking at the exposed surface.

PLSTIC STATE: The state of concrete from just after placing until the final setting of concrete.

POST-TENSIONING : Method of prestressing in which tendons are tensioned after concrete has hardened.

PRESTRESSED CONCRETE : Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

PRETENSIONING : Method of prestressing in which tendons are tensioned before concrete is placed.

SHRINKAGE LOSS : The loss of stress in the prestressing steel resulting from the shrinkage of concrete.

RELIABILITY: Ability of a structure to fulfill specified requirements during its designlife.

REMAINING SERVICE LIFE: Period from the point of inspection to the time when the structure is no longer useable, or does not satisfactorily perform the functions determined at the time of design.

REMEDIAL ACTION: Maintenance action carried out with the objective of arresting or slowing down the deterioration process, restoring or improving the performance of a structure, or reducing the danger of damage or injury to the users or any third party.

REPAIR: Remedial action taken with the objective of arresting or slowing down the deterioration of a structure, or reducing the possibility of damage to the users or third party.

RESTORABILTY (OR REPAIRABILTY): Ability of a structure to be repaired physically and economically when damaged under the effects of considered actions.

ROBUSTNESS (OR STRUCTURAL INSENSITIVITY): Ability of a structure to withstand damage by events like fire, explosion, impact, instability or consequences of human errors.

SAFETY: Ability of a structure to ensure that no harm would come to the users and to people in the vicinity of the structure under any action.

SEGREGATION: Separation of one or more constituent materials from the rest of the concrete, such as bleeding, aggregate blocking, etc.

SERVICE LIFE: The length of time from the completion of a structure until the time when it is no longer usable because of its failure to adequately perform its design functions.

SERVICEABILITY: Ability of a structure to provide adequate services or functionality in use under the effects of considered actions.

SETTLEMENT OF CONCRETE: Sinking of the concrete surface after placing due to bleeding and/or escaping of the entrapped and entrained air in the concrete.

SPECIAL CONCRETE: Concrete other than normal concrete including light weight concrete, roller compacted concrete, self-compacting concrete, fiber-reinforced concrete, anti-washout under water concrete, etc.

STIFF AND FLEXIBLE STRUCTURES: Stiff structures refer to those that are not sensitive to dynamic effects of wind, while flexible ones are those that are sensitive to such effects.

STRENGTHENING: Remedial action applied to a structure with the objective of restoring or improving its load bearing capacity to a level which is equal to, or higher than, the original design level.

STRESS AT TRANSFER : The stress in both the prestressing tendon and the concrete at the stage when the prestressing tendon is released from the prestressing mechanism.

SURFACE FINISHING: Action, such as troweling, applied to the exposed portion of concrete to obtain a neat surface.

TEMPERATURE CRACKING: Cracking caused by thermal stress which arises from differential temperatures in the concrete mass.

TENDON : Steel element such as wire, cable, bar, rod, or strand, or a bundle of such elements, used to impart prestress to concrete.

THRESHOLD LEVEL OF PERFORMANCE: Minimum acceptable level of performance of a structure.

TRANSFER: Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.

TRIBUTARY AREA: Area of building surface contributing to the force being considered, due to wind actions, and projected on a vertical plane normal to the wind direction.

TRANSMISSION LENGTH : The distance required at the end of a pretensioned tendon for developing the maximum tendon stress by bond.

ULTIMATE LIMIT STATE: Limit state for safety.

VARIABLE ACTION: Action due to a moving object on the structure as well as any load whose intensity is variable, including traffic load, wave load, water pressure, and load induced by temperature variation.

WOBBLE FRICTION: Friction caused by unintended deviation of prestressing sheath or duct from its specified profile.

WORKABILITY: The term expressing the ease with which concrete can be placed, compacted and filled.

9.3 Notations

a	= depth of equivalent rectangular stress block, mm
A	= area of the part of cross-section between flexural tension face and centre of gravity of gross section, mm ²
A_{ps}	= area of prestressed reinforcement in tension zone, mm ²
A_s	= area of nonprestressed tension reinforcement, mm ²
A'_s	= area of compression reinforcement, mm ²
b	= width of compression face of member, mm
d	= distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, mm
d'	= distance from extreme compression fiber to centroid of compression reinforcement, mm

d_b	= nominal diameter of bar, wire, or prestressing strand, mm
d_p	= distance from extreme compression fiber to centroid of prestressed reinforcement, mm
D	= dead loads, or related internal moments and forces
e	= base of Napierian logarithm
f'_c	= specified compressive strength of concrete, N/mm ²
f'_{ci}	= compressive strength of concrete at transfer of prestress, N/mm ²
f_d	= stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, N/mm ²
f_{pe}	= compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, N/mm ²
f_{pc}	= average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), N/mm ²
f_{ps}	= stress in prestressed reinforcement at nominal strength, N/mm ²
f_{pu}	= specified tensile strength of prestressing tendons, N/mm ²
f_{py}	= specified yield strength of prestressing tendons, N/mm ²
f_r	= modulus of rupture of concrete, N/mm ²
f_{se}	= effective stress in prestressed reinforcement (after allowance for all prestress)
f_y	= specified yield strength of nonprestressed reinforcement, N/mm ²
h	= overall thickness of member, mm
h_f	= overall thickness of flange of flanged section, mm
I	= moment of inertia of cross-section resisting externally applied factored loads, mm ⁴
k	= wobble friction coefficient per meter of prestressing tendon
ℓ	= length of span of two-way flat plates in direction parallel to that of the reinforcement being determined, mm
ℓ_x	= length of prestressing tendon element from jacking end to any point x, metre
L	= live loads, or related internal moments and forces
M_{cr}	= moment causing flexural cracking at section due to externally applied loads, kNm
M_{max}	= maximum factored moment at section due to externally applied loads, kNm
M_u	= factored moment at section, kNm
N_c	= tensile force in concrete due to unfactored dead load plus live load (D + L), kN
P_j	= prestressing tendon force at jacking end, kN

PI_p	=	Inherent or possessed performance index
PI_R	=	Inherent or possessed performance index
P_x	=	prestressing tendon force at any point x
s	=	spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, mm
V_c	=	nominal shear strength provided by concrete, kN
V_d	=	nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, kN
V_{cw}	=	nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, kN
V_d	=	shear force at section due to unfactored dead load, kN
V_i	=	factored shear force at section due to externally applied loads occurring simultaneously with M_{max} , kN
V_n	=	nominal shear strength, kN
V_p	=	vertical component of effective prestress force at section, kN
V_s	=	nominal shear strength provided by shear reinforcement, kN
V_u	=	factored shear force at section, kN
x	=	shorter overall dimension of rectangular part of cross-section
α	=	total angular change of prestressing tendon profile in radians from tendon jacking end to any point x
y	=	longer overall dimension of rectangular part of cross-section
y_t	=	distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension
β_1	=	factor defined in Sec 9.5.5(c)
γ_p	=	factor for type of prestressing tendon
	=	0.55 for f_{py}/f_{pu} not less than 0.80
	=	0.40 for f_{py}/f_{pu} not less than 0.85
	=	0.28 for f_{py}/f_{pu} not less than 0.90
μ	=	curvature friction coefficient
ρ	=	ratio of nonprestressed tension reinforcement = A_s/bd
ρ'	=	ratio of compression reinforcement = A'_s/bd
ρ_p	=	ratio of prestressed reinforcement = A_{ps}/bd_p
ϕ	=	strength reduction factor
ω	=	$\rho f_y/f'_c$
ω'	=	$\rho' f_y/f'_c$
ω_p	=	$\rho_p f_{ps}/f'_c$

$\omega_w, \omega_{pw}, \omega'_w =$ reinforcement indices for flanged sections computed for w , w_p , and w' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only.

For other symbols and units of quantities, reference shall be made to Chapter 6.

9.4 Analysis and design

9.4.1 Requirement

9.4.1.1 General

- (a) Prestressed members shall be designed for adequate strength in accordance with the provisions of this chapter.
- (b) Unless specifically excluded or superseded by the provisions of this chapter, all other relevant provisions of this code shall apply to prestressed concrete.
- (c) Design of prestressed members shall be based on strength and on the behavior at service conditions at all critical load stages during the life of the structure from the time of prestress is first applied.
- (d) Stress concentrations due to prestressing shall be considered in design.
- (e) Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length and rotations due to prestressing. Effects of temperature and shrinkage shall also be considered.
- (f) The possibility of buckling in a member between points where there is intermittent contact between prestressing steel and an oversized duct and buckling in thin webs and flanges shall be considered.
- (g) In computing section properties before bonding of prestressing steel, effect of loss of area due to open ducts shall be considered.
- (h) Thermal gradient and differential shrinkage shall be considered in composite construction using prestressed concrete members.
- (i) In evaluating the slenderness effects during lifting of slender beams, consideration shall be given to beam geometry, location of lifting points, method of lifting and tolerances in construction. All beams which are lifted on vertical or inclined slings shall be checked for lateral stability and lateral moment on account of tilting of beam. Reference may be made to specialist literature in this regard.

9.4.1.2 Design Assumptions

- (a) Strength design of prestressed members for flexure and axial loads shall be based on assumptions given in Sec 9.4.1.2 b. to g. and shall satisfy the applicable conditions of equilibrium and compatibility of strains.
- (b) Strains in steel and concrete shall be assumed to be directly proportional to the distance from the neutral axis except for Deep Beams.
- (c) Stress in nonprestressed reinforcement (if used) below f_y , shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y , stress in reinforcement shall be considered independent of strain and equal to f_y .
- (d) Maximum usable strain at extreme concrete compression fiber shall be assumed to be 0.003.
- (e) The relationship between concrete compressive stress distribution and concrete strain shall 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

- (f) Requirements of 9.4.1.2e. are satisfied by an equivalent rectangular concrete stress distribution defined by the following:
- i) 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 fiber of maximum compressive strain.
 - ii) Distance from the fiber of maximum strain to the neutral axis, c , measured in a direction perpendicular to the neutral axis.
 - iii) For f'_c between 17.5 and 28 MPa, β_1 shall be taken as 0.85. For f'_c above 28 MPa, β_1 shall be reduced linearly at a rate of 0.05 for each 7 MPa of strength in excess of 28 MPa, but β_1 shall not be taken less than 0.65.
- (g) For investigation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with the following assumptions:
- i) Strains vary linearly with depth through the entire load range.
 - ii) At cracked sections, concrete resists no tension.

9.4.1.3 Classification of prestressed concrete members

Prestressed concrete flexural members shall be classified as Class U (uncracked), Class T (transition) and Class C (cracked) based on extreme fiber stress in tension in the pre-compressed tensile zone as follows:

- (a) Class U: Permissible flexural tensile stresses $f_t \leq 0.62\sqrt{f'_c}$
- (b) Class T: $0.62\sqrt{f'_c} \leq f_t \leq 1.0\sqrt{f'_c}$
- (c) Class C: $f_t > 1.0\sqrt{f'_c}$
- (d) d. Prestressed two-way slab systems shall be designed as $f_t \leq 0.50\sqrt{f'_c}$

9.4.1.4 Shapes of beams and girders

For prestressed concrete non-composite beams/girders, the frequently used shapes are:

- (a) Symmetrical I-section, (b) Unsymmetrical I-section, (c) T-section,
- (d) Inverted T-section, (e) Box section, (f) solid rectangular section.

Commentary:

The suitability of selecting a particular shape will depend on the specific design requirement and economy of construction. In general, to achieve economy in steel and concrete, it is best to put the concrete near the extreme fibers of the compression flange. To suit this condition, I-section is the most natural choice. The inverted T-section may only be selected for composite construction when tension flange is precast and the compression flange is poured in place.

In choosing beam/girder shapes, due consideration should be given to the simplicity of formwork. When formwork is to be used once, it may constitute the major cost of beam/girder. Any irregular shape of beam/girder is generally unjustifiable on this ground. Rectangular solid sections are an obvious choice for flat slabs for floor and roof. Precast Tee or double Tee sections with a light concrete topping may be more economical choice for floor construction of multi-storied residential, commercial and factory buildings with moderate column spacing.

9.4.2 Material properties for design

9.4.2.1 Concrete

(a) Class

The Class of concrete is defined by the characteristic strength of concrete in cylinder at 28 days, f'_c . For example, Class 10 concrete indicates concrete with $f'_c = 10 \text{ N/mm}^2$. The classes of concrete commonly used are: Class 10, 15, 20, 25, 30, 35, 40, 45 and 50, although concrete in between these classes may also be used.

(b) Modulus of elasticity, E_c

Modulus of elasticity, E_c for concrete shall be permitted to be taken as $w_c^{1.5} 0.043 (f'_c)^{0.5}$ (in N/mm^2) for values of w_c between 1440 and 2560 kg/m^3 . For normal weight concrete, E_c may be permitted to be taken as $4700 (f'_c)^{0.5}$.

9.4.2.2 Reinforcing steel

a. Modulus of elasticity, E_s

Where it is not possible to ascertain the modulus of elasticity of reinforcing steel by test and from the manufacturer of steel, the modulus of elasticity of reinforcing steel may be permitted to be taken as $E_s = 200,000 \text{ N/mm}^2$.

9.4.2.3 Prestressing steel

(a) Modulus of elasticity, E_s

Where it is not possible to ascertain the modulus of elasticity of reinforcing and prestressing steel by test and from the manufacturer of steel, the values of E_s given in Table 9.4.1 may be used:

Table 9.4.1 Modulus of elasticity of prestressing steel and cold drawn wire

Type of steel	Modulus of elasticity, E_s (kN/mm^2)
Plain/indented cold-drawn wire	200
High tensile steel bars rolled or heat-treated	205
Strands	195

9.4.3 Nominal strengths of bonded reinforcement and of concrete at transfer

9.4.3.1 Bonded reinforcement

- Tensile stress at nominal strength of bonded reinforcement is limited to f_y for nonprestressed reinforcement and to f_{py} for prestressed reinforcement. Tensile stress at nominal strength of unbonded prestressed reinforcement for resisting tensile forces in the anchorage zone shall be limited to $f_{ps} = f_{se} + 70$.
- Except for concrete confined within spirals or hoops providing confinement equivalent to that corresponding to Eq. (9.4.3-1), compressive strength in concrete in the general zone shall be limited to $0.7\lambda f'_{ci}$.

Commentary:

Eq. (9.4.3-1) is given below.

$$\rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \quad (9.4.3-1)$$

where the value of f_{yt} used in Eq. (9.4.3-1) shall not exceed 700 N/mm^2 . For f_{yt} greater than 420 N/mm^2 normal lap splices as shown in Chapter 6 shall not be applied.

9.4.3.2 Concrete strength at transfer

Compressive strength of concrete at time of post-tensioning shall be specified in the contract documents. Unless oversize anchorage devices are sized to compensate for the lower compressive strength or the prestressing steel is stressed to no more than 50 percent of the final prestressing force, prestressing steel shall not be stressed until compressive strength of concrete as indicated by tests consistent with the curing of the member, is at least 28 N/mm^2 for multi-strand tendons or at least 17 N/mm^2 for single-strand or bar tendons.

9.4.4 Serviceability Requirements – Flexural Members

9.4.4.1 Stresses in concrete immediately after prestress transfer

Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses occur) are as follows:

- (a) Extreme fiber stress in compression except as permitted in (b) shall not exceed $0.60f'_{ci}$
- (b) Extreme fiber stress in compression at ends of simply support members shall not exceed... $0.70f'_{ci}$
- (c) Where computed concrete tensile strength, f_t , exceeds $0.5\sqrt{f'_c}$ at ends of simply supported members, or $0.25\sqrt{f'_c}$ at other locations, additional bonded reinforcement shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

9.4.4.1.1 Stresses in concrete at service loads and reinforcement spacing

9.4.4.1.2 Allowable stresses in concrete

For Class U and Class T prestressed flexural members, stresses in concrete at service loads (based on uncracked section properties and after allowance for all prestress losses) shall not exceed the following:

- (a) Extreme fiber stress in compression due to prestress plus sustained load $0.45f'_c$
- (b) Extreme fiber stress in compression due to prestress plus total load..... $0.60f'_c$
- (c) Permissible stresses in 9.4.3.2 and 9.4.4.1 shall be permitted to be exceeded if shown by test or analysis that performance will not be impaired.

9.4.4.1.3 Reinforcement spacing

- (a) For Class C prestressed flexural members not subject to fatigue or to aggressive exposure, the spacing of bonded reinforcement nearest the extreme tension face shall not exceed that for normal Reinforced Concrete, as given below:

$$s = 380 (280/f_s) - 2.5 c_c \quad (9.4.4-1)$$

but not greater than $300 (280/f_s)$, where c_c is the least distance from the surface of reinforcement or prestressing steel to the tension face. If there is only one bar or wire nearest to the extreme tension face, s used in the above equation is the width of the extreme tension face.

- (b) Calculated stress f_s in reinforcement closest to the tension face at service loads shall be computed based on the unfactored moment. It shall be permitted to take f_s as $2/3f_y$.
- (c) For structures subject to fatigue or exposed to corrosive environments, investigations, judgment and precautions are required.
- (d) The spacing requirements shall be met by nonprestressed reinforcement and bonded tendons. The spacing of bonded tendons shall not exceed 2/3rd of the maximum spacing permitted for nonprestressed reinforcement.
- (e) Where both reinforcement and bonded tendons are used to meet the spacing requirement, the spacing between a bar and a tendon shall not exceed 5/6 of that permitted by 9.4.3.2.2 a., b. and c.
- (f) In applying Eq. 9.4.4-1 to prestressing tendons, Δf_{ps} shall be substituted for f_s , where Δf_{ps} shall be taken as the calculated stress in the prestressing steel at service loads based on a cracked section analysis minus the decompression stress f_{dc} . It shall be permitted to take f_{dc} equal to the effective stress in the prestressing steel f_{se} . See also 9.4.4.2.2g. below.
- (g) In applying Eq. (9.4.4-1) to prestressing tendons, the magnitude of Δf_{ps} shall not exceed 250 N/mm^2 . When Δf_{ps} is less than or equal to 140 N/mm^2 , the spacing requirements of 9.4.4.2.2 a. and b. shall not apply.

- (h) Where h of a beam exceeds 900 mm, the area of longitudinal skin reinforcement consisting of untensioned reinforcing steel or bonded tendons shall be uniformly distributed along both side faces of the member. Skin reinforcement shall extend for a distance of $h/2$ from the tension face. The spacing s shall be as provided in 9.4.4.2.2, where c_c is the least distance from the surface of the skin reinforcement or prestressing steel to the side face. It shall be permitted to include such reinforcement or prestressing steel to the side face. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires.

9.4.5 Permissible stresses in prestressing steel

Tensile stress in prestressing tendons shall not exceed the following:

- (a) Due to prestressing steel jacking force $0.94f_{py}$
but not greater than the lesser of $0.80f_{pu}$ and the maximum value recommended by the manufacturer of prestressing steel or anchorage devices.
- (b) Immediately after prestress transfer..... $0.82f_{py}$
but not greater than $0.74f_{pu}$.
- (c) Post-tensioning tendons, at anchorage devices and couplers, immediately after force transfer..... $0.70f_{pu}$

9.4.6 Losses of prestress

To determine effective stress in the prestressing steel, f_{se} , allowance for the following sources of loss of prestress shall be considered:

9.4.6.1 Immediate losses

- (a) Loss due to elastic shortening of concrete;
- (b) Loss due to prestressing steel seating at transfer (Anchorage slip);
- (c) Loss due to friction (for post-tensioned concrete only).

9.4.6.2 Long-term losses

- (a) Loss due to relaxation of prestressing steel stress;
- (b) Loss due to creep of concrete;
- (c) Loss due to shrinkage of concrete.

Unless otherwise determined by actual tests, allowance for these losses shall be made in accordance with the provisions of Sec 9.4.6.3 through 9.4.6.8.

9.4.6.3 Loss due to elastic shortening of concrete

- (a) The loss of prestress due to immediate elastic shortening of adjacent concrete upon transfer of initial prestress shall be calculated as specified in this section. For pretensioning, the loss of prestress in the tendons at transfer shall be calculated on a modular ratio basis using the stress in the adjacent concrete.
- (b) For members with post-tensioned tendons which are not stressed simultaneously, there is a progressive loss of prestress during transfer due to the gradual application of the prestressing forces. This loss of prestress shall be calculated on the basis of half the product of the stress in the concrete adjacent to the tendons averaged along their lengths and the modular ratio. Alternatively, the loss of prestress may be exactly computed based on the sequence of tensioning.

9.4.6.4 Loss Due to prestressing steel seating at transfer (Anchorage slip)

- a. Any loss of prestress which may occur due to slip of wire or strand during anchoring or due to straining of the anchorage shall be allowed for in the design. Necessary additional elongation may be provided for at the time of tensioning to compensate for this loss.

9.4.6.5 Loss due to friction (for post-tensioned tendons only)

- (a) The design shall take into consideration all losses in prestress that may occur during tensioning due to friction between the post-tensioning tendons and the surrounding concrete or any fixture attached to the steel or concrete.
- (b) The value of prestressing force P_x at a distance ℓ_x metres from the jacking end and acting in the direction of the tangent to the curve of the cable, shall be calculated from the relation:

$$P_x = P_j e^{-(K\ell_x + \mu\alpha)} \quad (9.4.5-1)$$

When $(K\ell_x + \mu\alpha)$ is greater than 0.3, P_x may be computed from

$$P_x = \frac{P_j}{1 + K\ell_x + \mu\alpha} \quad (9.4.5-2)$$

For use in Eq. (9.4.5-1) and Eq. (9.4.5-2), the values of wobble friction coefficient K and curvature friction coefficient μ shall be experimentally determined or obtained from the tendon manufacturer, and verified during tendon stressing operations.

- (c) In absence of test results or manufacturer's recommendation, the following values of μ and K shown in Table 9.4.2 may be taken as a guide:

Table 9.4.2 Friction Coefficients (K & μ) for post-tensioned tendons

			coefficient, K per meter	Curvature coefficient, μ per radian
Grouted Tendons in metal sheathing		Wire tendons	0.0033-0.0049	0.15-0.25
		High-strength bars	0.0003-0.0020	0.08-0.30
		7-wire strand	0.0016-0.0066	0.15-0.25
Unbonded tendons	Mastic coated	Wire tendons	0.0033-0.0066	0.05-0.15
		7-wire strand	0.0033-0.0066	0.05-0.15
	Pre- greased	Wire tendons	0.001-0.0066	0.05-0.15
		7-wire strand	0.001-0.0066	0.05-0.15

Values of wobble and curvature friction coefficients used in design shall be shown on design drawings.

- (e) The effect of reverse friction shall be taken into consideration in such cases where the initial tension applied to a prestressing tendon is partially released and action of friction in the reverse direction causes significant alteration in the distribution of stress along the length of the tendon.
- (f) Where loss of prestress in a member occurs due to connection of member to adjoining construction, such loss of prestress shall be allowed for in design.

9.4.6.6 Loss due to relaxation of prestressing steel stress

- (a) The relaxation losses in prestressing steel shall be determined from experiments. When experimental values are not available, the relaxation losses, considering normal relaxation steel, may be assumed as given in Table 9.4.3.

Table 9.4.3 Relaxation Losses for prestressing steel at 1000 hours at 27°C

Initial Stress	Relaxation Loss
	N/mm ²
0.5 fpu	0
0.6 fpu	35
0.7 fpu	70
0.8 fpu	90

For tendons at higher temperature or subject to large lateral loads, greater relaxation losses may be allowed, subject to the advice of the metallurgy specialist.

- (b) No reduction in the value of the relaxation losses should be made for a tendon with a load equal to or greater than the relevant jacking force that has been applied for a short duration prior to the anchoring of the tendon.

9.4.6.7 Loss due to creep of concrete

- (a) Creep occurs due to superimposed permanent dead load added to the member after it has been prestressed. Creep of concrete may be assumed to be proportional to the stress provided the stress in concrete does not exceed 40 per cent of its compressive strength.
- (b) In the absence of test data, the ultimate creep strain may be estimated from the following values of creep coefficient, which is the ratio of the ultimate creep strain to the elastic strain at the age of loading. Table 9.4.4 shows the values at different days.

Table 9.4.4 Creep coefficient of concrete

Age at Loading	Creep coefficient
7 days	2.2
28 days	1.6
1 year	1.1

- (c) The ultimate creep strain estimated as above does not include the elastic strain. For the calculation of deformation at some stage before the total creep is reached, it may be assumed that 50 per cent of the total creep takes place in the first month after loading and about 75 per cent of the total creep takes place in the first six months after loading. For post-tensioning the creep coefficients shall be taken as 80% of those given here.
- (d) The loss of prestress due to creep of concrete shall be determined for all the permanently applied loads including the prestress. Loss due to stresses of short duration including live load and erection stresses may be ignored.
- (e) The loss of prestress due to creep of concrete shall be obtained as the product of the modulus of elasticity of the prestressing steel and the ultimate creep strain of the concrete fiber integrated along the centre-line of the prestressing steel over its entire length.
- (f) The total creep strain during any specific period shall be assumed to be the creep strain due to sustained stress equal to the average of the stresses at the beginning and end of the period.

9.4.6.8 Loss due to shrinkage of concrete

- (a) In the absence of test data, the approximate value of shrinkage strain in concrete for design purposes shall be assumed as follows:

For pretensioning : 0.0003

For post-tensioning : $\frac{0.0002}{\log_{10}(t+2)}$

where, t = age of concrete at transfer in days.

- (b) For the calculation of deformation of concrete at some stage before the maximum shrinkage occurs it may be assumed that 50 per cent of the shrinkage takes place during the first month

and about 75 per cent of the shrinkage takes place in the first six months after drying of concrete starts.

- (c) The loss of prestress due to shrinkage of concrete shall be obtained as the product of the modulus of elasticity of steel and the shrinkage strain of concrete.

9.4.7 Control of Deflection

9.4.7.1 For prestressed concrete flexural members, designed in accordance with the provisions of this chapter, immediate deflection shall be computed by usual methods or formulas for elastic deflections, and the moment of inertia of gross concrete section, I_g , shall be permitted to be used for Class U flexural members.

9.4.7.2 For Class C and Class T flexural members, deflection calculations shall be based on cracked transformed section analysis. It shall be permitted to base calculations on an effective moment of inertia, I_e as given in Eq. 9.4.7-1.

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \quad (9.4.7-1)$$

$$\text{Where, } M_{cr} = \frac{f_r I_g}{y_t} \quad \text{and} \quad f_r = 0.62 \lambda \sqrt{f'_c}$$

Deflection computed in accordance with 9.4.7.1 shall not exceed the limits stipulated in 6.5.9.4.

9.4.7.3 Additional long-term deflection of prestressed 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.

9.4.8 Flexural Strength

9.4.8.1 Design moment strength of flexural members shall be computed by the strength methods of the Code. For prestressing steel, f_{ps} shall be substituted for f_y in strength computations.

9.4.8.2 As an alternative to a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} shall be permitted to be used if f_{se} is not less than $0.5f_{pu}$.

(a) For members with bonded tendons

$$f_{ps} = f_{pu} \left\{ 1 - \frac{\gamma_p}{\beta_1} \left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p} (\omega - \omega') \right] \right\} \quad (9.4.8-1)$$

where ω is $\rho_p f_y / f'_c$, ω' is $\rho'_p f_y / f'_c$, and γ_p is 0.55 for f_{py} / f_{pu} not less than 0.80; 0.40 for f_{py} / f_{pu} not less than 0.85; and 0.28 for f_{py} / f_{pu} not less than 0.90.

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. (9.4.8-1), the term shall be taken not less than 0.17 and d' shall be no greater than d_p .

$$\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p} (\omega - \omega') \right]$$

(b) For members with unbonded tendons and with a span-to-depth ratio of 35 or less:

$$f_{ps} = f_{se} + 70 + \frac{f'_c}{100 \rho_p} \quad (9.4.7-2)$$

but f_{ps} in Eq. (9.4.7.2) shall not be taken greater than the lesser of f_{py} and $(f_{se} + 420)$.

- c. For members with unbonded tendons and with a span-to-depth ratio greater than 35:

$$f_{ps} = f_{se} + 70 + \frac{f'_c}{300\rho_p} \quad (9.4.7-3)$$

but f_{ps} in Eq. (9.4.7.3) shall not be taken greater than the lesser of f_{py} and $(f_{se} + 210)$

- 9.4.8.3 Nonprestressed reinforcement conforming to 9.4.4.2.2, if used with prestressing steel, shall be permitted to be considered to contribute to the tensile force and to be included in moment strength computations at a stress equal to f_y . Other nonprestressed reinforcement shall be permitted to be included in strength computations only if a strain compatibility analysis is performed to determine stresses in such reinforcement.

9.4.9 Limits for flexural reinforcement

- 9.4.9.1 Prestressed concrete sections shall be classified as either tension-controlled, transition, or compression-controlled sections, in accordance with a. and b. below.

- (a) Sections are compression-controlled if the net tensile strain in the extreme tension fiber ϵ_t , is equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 420 reinforcement, and for all prestressed reinforcement, it shall be permitted to set the compression-controlled strain limit to 0.002.
- (b) Sections are tension-controlled if the net tensile strain in the extreme tension steel, ϵ_t , is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with ϵ_t between the compression-controlled strain limit and 0.005 constitute a tension region between the compression-controlled and tension-controlled sections. Appropriate strength reduction factor, ϕ , from 9.4.9.2 shall apply.

- 9.4.9.2 The appropriate strength reduction factor, ϕ , shall apply as given in a. to f. below.

(a) Tension-controlled sections	0.90
(b) For compression-controlled sections	
i. Members with spiral reinforcement as defined in 9.4.9.4	0.75
ii. Other reinforced members	0.65
(c) Shear and torsion	0.75
(d) Post-tensioned anchorage zones	0.85
(e) Strut and tie models	0.75
(f) Flexural sections in pre-tensioned members where strand embedment length is less than the development length	
i. From the end of the member to the end of the transfer length	0.75
ii. From the end of the transfer length to the end of the development length, ϕ shall be taken as	0.75 to 0.90

Where bonding of the strand does not extend to the end of the member, strand embedment shall be assumed to begin at the end of the debonded length.

- 9.4.9.3 Total amount of prestressed and nonprestressed reinforcement in members with bonded prestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load

computed on the basis of the modulus of rupture f_r , as given in 9.4.6.2. This provision shall be permitted to be waived for flexural members with shear and flexural strength at least twice that required by 9.4.8.

9.4.9.4 Volumetric spiral reinforcement ratio, ρ_s , shall be not less than the value given by

$$\rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f_c'}{f_{yt}} \quad (9.4.9-1)$$

where the value of f_{yt} in Eq. (9.4.9-1) shall not exceed 700 N/mm². For f_{yt} greater than 420 N/mm² where lap splices shall not exceed in accordance with 9.4.9.4 a. this shall not be used.

(a) Spiral reinforcement shall be spliced, if needed, by any one of the following methods:

Lap splices not less than the larger of 300 mm and the length indicated in one of (1) through (5) below:

- i) deformed uncoated bar or wire $48d_b$
- ii) plain uncoated bar or wire $72d_b$
- iii) epoxy-coated deformed bar or wire $72d_b$
- iv) plain uncoated bar or wire with a standard stirrup or tie hook in accordance with 9.4.8.5 c. at ends of lapped spiral reinforcement.

(b) The term “standard hook” as used in this code shall mean one of the following:

- i) 180-degree bend plus $4d_b$ extension, but not less than 65 mm at free end of bar.
- ii) 90-degree bend plus $12d_b$ extension at free end of bar.

(c) For stirrup and tie hooks

- i) No. 16 bar and smaller, 90-degree bend plus $6d_b$ extension at free end of bar; or
- ii) No. 19, No. 22 bar and No. 25 bar, 90-degree bend plus $12d_b$ extension at free end of bar; or
- iii) No. 25 bar and smaller, 135-degree bend plus $6d_b$ extension at free end of bar.

9.4.9.5 Part or all of the bonded reinforcement consisting of bars or tendons shall be provided as close as practicable to the tension face in prestressed flexural members. In members prestressed with unbonded tendons, the minimum bonded reinforcement consisting of bars or tendons shall be as required by 9.4.10.

9.4.10 Minimum bonded reinforcement

9.4.10.1 A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded tendons as required by 9.4.10.2 and 9.4.10.3.

9.4.10.2 Except as provided in 9.4.10.1, minimum area of bonded reinforcement shall be computed by

$$A_s = 0.004A_{ct} \quad (9.4.10-1)$$

where A_{ct} is area of that part of cross section between the flexural tension face and center of gravity of gross section.

(a) Bonded reinforcement required by Eq. (9.4.10-1) shall be uniformly distributed over pre-compressed tensile zone as close as practicable to extreme tension fiber.

(b) Bonded reinforcement shall be required regardless of service load stress conditions.

9.4.10.3 For two-way flat slab systems, minimum area and distribution of bonded reinforcement shall be as required in a., b., and c below.

(a) Bonded reinforcement shall not be required in positive moment areas where f_t , the extreme fibre stress in tension in the precompressed tensile zone at service loads (after allowance for all prestress losses), does not exceed $0.17vf'_c$.

(b) In positive moment areas where computed tensile stress in concrete at service load exceeds $0.17vf'_c$ minimum area of bonded reinforcement shall be computed by

$$A_s = N_c / 0.5f_y \quad (9.4.10-2)$$

where the value of f_y used in Eq. (9.4.10-2) shall not exceed 420 MPa. Bonded reinforcement shall be uniformly distributed over precompressed tensile zone as close as practicable to the extreme tension fiber.

(c) In negative moment areas at column supports, the minimum area of bonded reinforcement A_s in the top of the slab in each direction shall be computed by

$$A_s = 0.00075A_{cf} \quad (9.4.10-3)$$

where, A_{cf} is the larger gross cross-sectional area of the slab-beam strips in two orthogonal equivalent frames intersecting at a column in a two-way slab.

9.4.10.4 Bonded reinforcement required by Eq. (9.4.10-3) shall be distributed between lines that are 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.

9.4.10.5 Minimum length of bonded reinforcement required by 9.4.10.2 and 9.4.10.3 shall be as required in 9.4.10.5 a., b., and c.

(a) In positive moment areas, minimum length of bonded reinforcement shall be one-third the clear span length, l_n , and centered in positive moment area.

(b) In negative moment areas, bonded reinforcement shall extend one-sixth the clear span, l_n , on each side of support.

(c) Where bonded reinforcement is provided for ϕM_n in accordance with 9.4.7.8, or for tensile stress conditions in accordance with 9.4.9.3 b., minimum length also shall conform to provisions of Chapter 6.

9.4.11 Statically indeterminate structures

9.4.11.1 Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

9.4.11.2 Performance at service load conditions shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces induced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

9.4.11.3 Moments 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. Adjustment of the sum of these moments shall be permitted as allowed in 9.4.11.4.

9.4.11.4 Redistribution of moments in continuous prestressed flexural members shall be as follows:

(a) Where bonded reinforcement is provided at supports in accordance with 9.4.11, it shall be permitted to decrease negative or positive moments calculated by elastic theory for any assumed loading, in accordance with 9.4.11.4 b., c. and d. below.

(b) Except where approximate values for moments are used, it shall be permitted to decrease factored moments calculated by elastic theory at sections of maximum negative or maximum positive moment in any span of continuous flexural members for any assumed loading arrangement by not more than $1000\epsilon_t$ percent, with a maximum of 20 percent.

- (c) Redistribution of moment shall be made only when ϵ_t is equal to or greater than 0.0075 at the section at which moment is reduced.
- (d) The reduced moment shall be used for calculating redistributed moments at all other sections within the spans. Static equilibrium shall be maintained after redistribution of moments for each loading arrangement.

9.4.12 Compression members — Combined flexure and axial load

9.4.12.1 Prestressed concrete members subject to combined flexure and axial load, with or without non-prestressed reinforcement, shall be proportioned by the strength design methods of this Code. Effects of prestress, creep, shrinkage, and temperature change shall be included.

9.4.12.2 Limits for reinforcement of prestressed compression members shall be as follows:

- (a) Members with average compressive stress in concrete less than 1.6 N/mm^2 , due to effective prestress force only, shall have minimum reinforcement in accordance with Chapter 6.
- (b) Except for walls, members with average compressive stress in concrete due to effective prestress force only, equal to or greater than 1.6 N/mm^2 shall have all tendons enclosed by spirals or lateral ties in accordance with c. through g.
- (c) Spirals shall conform to Chapter 6.
- (d) Lateral ties shall be at least No. 10 in size or welded wire reinforcement of equivalent area, and shall be spaced vertically not to exceed 48 tie bar or wire diameters, or the least dimension of the compression member.
- (e) Ties shall be located vertically not more than half a tie spacing above top of footing or slab in any story, and not more than half a tie spacing below the lowest horizontal reinforcement in members supported above.
- (f) Where beams or brackets frame into all sides of a column, ties shall be terminated not more than 75 mm below lowest reinforcement in such beams or brackets.
- (g) For walls with average compressive stress in concrete due to effective prestress force only equal to or greater than 1.6 N/mm^2 , minimum reinforcement required by Chapter 6 shall not apply where structural analysis shows adequate strength and stability.

9.4.13 Slab systems

9.4.13.1 Factored moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with provisions of 6.5 or by more detailed design procedures.

9.4.13.2 ϕM_n of prestressed slabs with loads and load combinations required by Chapter 2 and 6 at every section shall be greater than or equal to M_u considering 9.4.11.3 and 9.4.11.4. ϕV_n of prestressed slabs at columns required by Chapter 6 shall be greater than or equal to V_u .

9.4.13.3 At service load conditions, all serviceability limitations, including limits on deflections, shall be met, with appropriate consideration of the factors listed in 9.4.11.2.

9.4.13.4 For uniformly distributed loads, spacing of tendons or groups of tendons in at least one direction shall not exceed the smaller of eight times the slab thickness and 1.5 m. Spacing of tendons also shall provide a minimum average effective prestress of 0.9 N/mm^2 on the slab section tributary to the tendon or tendon group. For slabs with varying cross section along the slab span, either parallel or perpendicular to the tendon or tendon group, the minimum average effective prestress of 0.9 N/mm^2 is required at every cross section tributary to the tendon or tendon group along the span. Concentrated loads and opening in slabs shall be considered when determining tendon spacing.

- 9.4.13.5 In slabs with unbonded tendons, bonded reinforcement shall be provided in accordance with 9.4.9.3 and 9.4.9.5.

Except as permitted in 9.4.13.6, in slabs with unbonded tendons, a minimum of two 12.7 mm diameter or larger, seven-wire post-tensioned strands shall be provided in each direction at columns, either passing through or anchored within the region bounded by the longitudinal reinforcement of the column. Outside column and shear cap faces, these two structural integrity tendons shall pass under any orthogonal tendons in adjacent spans. Where the two structural integrity tendons are anchored within the region bounded by the longitudinal reinforcement of the column, the anchorage shall be located beyond the column centroid and away from the anchored span.

- 9.4.13.6 Prestressed slabs not satisfying 9.4.13.5 shall be permitted provided they contain bottom reinforcement in each direction passing within the region bounded by the longitudinal reinforcement of the column and anchored at exterior supports as required by bar detailing requirement of slabs given in Chapter 6. The area of bottom reinforcement in each direction shall be not less than 1.5 times that required by Eq. (9.4.13-1) as given below.

$$A_{s, \min} = \frac{0.25\sqrt{f'_c}}{f_y} b_w d \quad (9.4.13-1)$$

and not less than $2.1b_w d/f_y$, where b_w is the width of the column face through which the reinforcement passes. Minimum extension of these bars beyond the column or shear cap face shall be equal to or greater than the bar development length required by Chapter 6.

- 9.4.13.7 In lift slabs, bonded bottom reinforcement shall be detailed in accordance with 9.4.13.8.
- 9.4.13.8 In slabs with shear heads and in lift slab construction where it is not practical to pass the bottom bars, required by bar detailing requirement of Chapter 6, at least two bonded bars or wires in each direction shall pass through the shear head or lifting collar as close to the column as practicable and be continuous or spliced with a Class A splice. At the exterior columns, the reinforcement shall be anchored the shear head or lifting collar.

9.4.14 Post-tensioned tendon anchorage zones

9.4.14.1 Division into zones

The anchorage zone shall be considered as composed of two zones as described below and shown in Fig. 9.4.14-1.

- The local zone is the rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement;
- The general zone is the anchorage zone beyond the local zone.

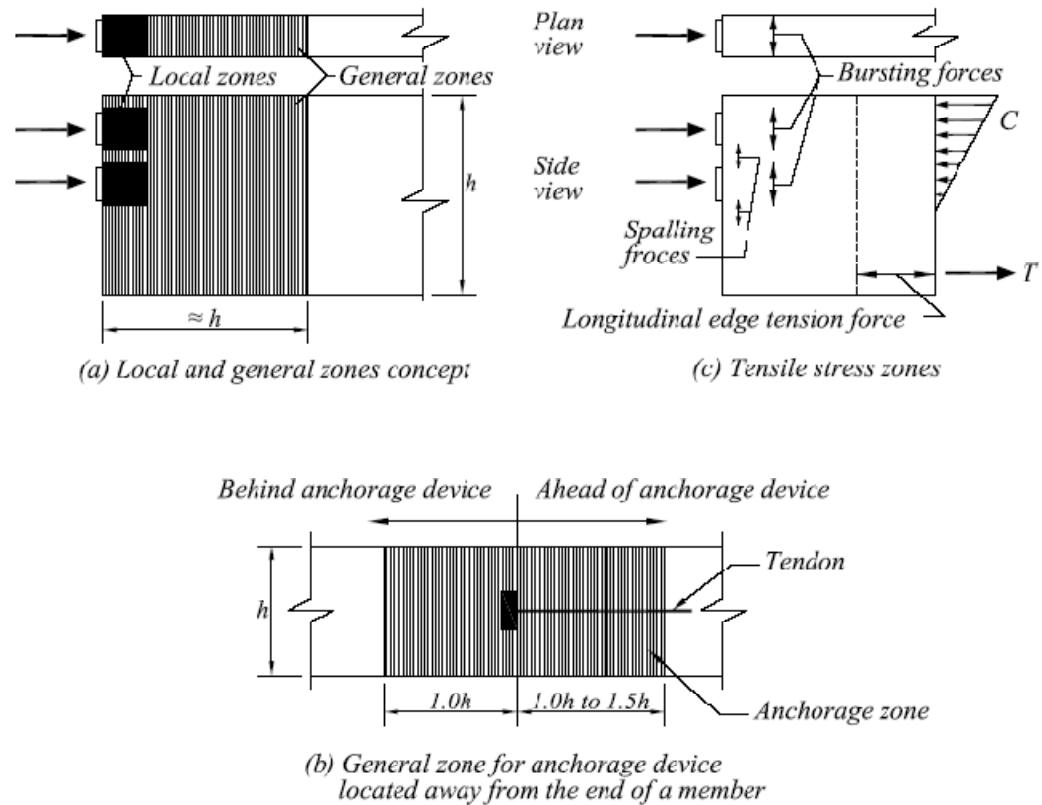


Fig.9.4.14-1 Anchorage zones

9.4.14.2 Local zone

- Design of local zones shall be based upon the factored prestressing force, P_{pu} , and the requirements of 9.4.14.2 b. and c. below.
- For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum steel jacking force.
- While 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 the strength reduction factor, ϕ , for post-tensioned anchorage zone this shall be taken as 0.85.
- Local-zone reinforcement shall be provided where required for proper functioning of the anchorage device.

9.4.14.3 General zone

- Design of general zones shall be based upon the factored prestressing force, P_{pu} , and the requirements of 9.4.14.3 b. and c.
- General-zone reinforcement shall be provided where required to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices. Effects of abrupt change in section shall be considered.
- The general zone requirements of 9.4.14.3b. are satisfied by 9.4.3, 9.4.14.4, and 9.4.14.5 and whichever one of 9.4.15.2 or 9.4.15.3 or 9.4.16.3 is applicable.

9.4.14.4 Design methods

The following methods shall be permitted for the design of the general zones of the prestressed components provided that the specific procedures used result in prediction of strength in substantial agreement with results of comprehensive tests:

- a. Equilibrium-based plasticity models (strut-and-tie models);
- b. Linear stress analysis (including finite element analysis or equivalent); or
- c. Simplified equations where applicable.
- d. Simplified equations shall not be used where member cross sections are nonrectangular, where discontinuities in or near the general zone cause deviations in the force flow path, where minimum edge distance is less than 1-1/2 times the anchorage device lateral dimension in that direction, or where multiple anchorage devices are used in other than one closely spaced group.
- e. The stressing sequence shall be considered in the design and specified on the design drawings.
- f. Three-dimensional effects shall be considered in design and analyzed using three-dimensional procedures or approximated by considering the summation of effects for two orthogonal planes.
- g. For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $0.35A_{ps}f_{pu}$ into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage devices and shall be fully developed both behind and ahead of the anchorage devices.
- h. Where tendons are curved in the general zone, except for mono-strand tendons in slabs or where analysis shows reinforcement is not required, bonded reinforcement shall be provided to resist radial and splitting forces.
- i. Except for mono-strand tendons in slabs or where analysis shows reinforcement is not required, minimum reinforcement with a nominal tensile strength equal to 2 percent of each factored prestressing force shall be provided in orthogonal directions parallel to the back face of all anchorage zones to limit spalling.
- j. Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

9.4.14.5 Detailing requirements

Selection of reinforcement sizes, spacing, cover, and other details for anchorage zones shall make allowances for tolerances on the bending, fabrication, and placement of reinforcement, for the size of aggregate, and for adequate placement and consolidation of the concrete.

9.4.15 Design of anchorage zones for monostrand or single $\phi 16$ mm bar tendons

9.4.15.1 Local zone design

Monostrand or single $\phi 16$ mm or smaller diameter bar anchorage devices and local zone reinforcement shall meet the requirements of ACI 423.7 or the special anchorage device requirements of [9.4.16.2](#).

9.4.15.2 General zone design for slab tendons

- (a) For anchorage devices of 12.7 mm or smaller diameter strands in normal weight concrete slabs, minimum reinforcement meeting the requirements of [9.4.15.2 b. and c.](#) shall be provided unless a detailed analysis satisfying [9.4.14.4](#) shows such reinforcement is not required.
- (b) Two horizontal bars at least No. 13 in size shall be provided parallel to the slab edge. They shall be permitted to be in contact with the front face of the anchorage device and shall be within a distance of $h/2$ ahead of each device. Those bars shall extend at least 150 mm either side of the outer edges of each device.
- (c) If the center-to-center spacing of anchorage devices is 300 mm or less, the anchorage devices shall be considered as a group. For each group of six or more anchorage devices, $n + 1$ hairpin bars or closed stirrups at least No. 10 in size shall be provided, where n is the number of anchorage devices. One hairpin bar or stirrup shall be placed between each anchorage device and one on each side of the group. The hairpin bars or stirrups shall be placed with the legs extending into the slab perpendicular to the edge. The center portion of the hairpin bars or

stirrups shall be placed perpendicular to the plane of the slab from $3h/8$ to $h/2$ ahead of the anchorage devices.

- (d) For anchorage devices not conforming to 9.4.15.1, minimum reinforcement shall be based upon a detailed analysis satisfying 9.4.14.4.

9.4.15.3 General zone design for groups of monostrand tendons in beams and girders

Design of general zones for groups of monostrand tendons in beams and girders shall meet the requirements of 9.4.13.3, 9.4.14.2 or 9.4.14.3 or 9.4.15.3.

9.4.16 Design of anchorage zones for multi-strand tendons

9.4.16.1 Local and general zone design

Basic multistrand anchorage devices and the related local and general zone reinforcement shall meet the requirements of AASHTO "LRFD Bridge Design Specifications (SI), 2007", Articles 5.10.9.6, Approximate Stress Analysis and Design, and 5.10.9.7, Design of Local Zones.

9.4.16.2 Special anchorage devices

- (a) AASHTO "LRFD Bridge Design Specifications (SI), 2007", Articles 5.10.9.7.3, Special Anchorage Devices requires that special anchorage devices that do not satisfy the requirements specified in 9.4.16.1, they have been tested by an independent testing agency acceptable to the Engineer and have met the acceptance criteria specified in Articles 10.3.2 and 10.3.2.3.10 of AASHTO LRFD Bridge Construction Specifications.
- (b) Where special anchorage devices are to be used, supplemental skin reinforcement shall be furnished in the corresponding regions of the anchorage zone, in addition to the confining reinforcement specified for the anchorage device. This supplemental reinforcement shall be similar in configuration and at least equivalent in volumetric ratio to any supplementary skin reinforcement used in the qualifying acceptance tests of the anchorage device.

9.4.17 Cold Drawn Low Carbon Wire Prestressed Concrete (CWPC)

CWPC (Cold drawn wire prestressed concrete) is termed as prestressed concrete technology of Chinese pattern. This technology is a modification of conventional prestressed concrete. In the conventional prestressed concrete high strength wire is used as reinforcement while in Chinese pattern cold drawn low carbon mild steel wire is used as such this technology is named as cold drawn wire prestressed concrete. In short it is termed as CWPC. CWPC technology is a process whereby cold drawn low carbon steel wire has been adopted as reinforcement for pre-fabricated prestressed concrete members of medium and small size as produced by pre tensioning method. In the other hand large size structural members are produced by conventional prestressed concrete.

The main features and advantages of CWPC technology can be summarised as follows:

- (a) **Availability (Availability of materials):** The raw material of cold drawn wire is made from low carbon mild steel which can be supplied by the local mills. The tensioning process of cold-drawn wire and production of pre-cast members are also simple and very easy to handle.
- (b) **Simplicity (Simplicity of equipment and devices for production):** The cold process of low carbon mild steel and prefabrication process of members are done using simple equipments and devices. The precise and large sized equipments are not necessary. The production techniques of manufacturing members are rather simple.
- (c) **Quality (Good in quality):** The members so manufactured have high crack resistance and stiffness. After pre-tensioning no crack would occur under the service load, thus the wires within the concrete members are well protected. In Contrast to conventional reinforced concrete members under the same service conditions, they have comparatively high durability to ensure long term quality.
- (d) **Economy (Low cost):** The cold drawn low carbon steel wire used for prestressing is made of ordinary hot-rolled carbon steel coil rod. This is processed at room temperature through a special wire drawing die. The low carbon coil rods are manufactured by the steel mills; the wires are processed at the construction site or in a prefabrication plant; or are supplied by the cold drawn wire plants as ready made products. By cold drawing the low carbon rod into wires the strength is enhanced about twice as

much as that of the coil rod. This reduces the amount of steel required in prefabricating prestressed concrete members.

- (e) Therefore, in comparison with conventional reinforced concrete reinforced with common carbon steel, a prestressed concrete member reinforced with cold drawn wire would have saving of steel consumption by 30-50%. Furthermore, since prestressed concrete members have high stiffness a reduction of cross section of members is possible. A considerable amount of concrete can also be saved and hence work including transportation, handling and erection can be reduced.
- (f) **Light weightedness (Light in weight):** As already mentioned that the stiffness of prestressed concrete members may be enhanced, the dimension of it's cross-section can be reduced crosspondingly. These result not only reduction of concrete volume but also its dead weight which is estimated as 10-30%.

9.4.17.1 Materials

Basically the materials used in CWPC technology are steel and concrete.

- (a) **Steel:** steel used for CWPC is obtained by cold drawing. Cold drawing as already mentioned is a process of reducing the diameter of the coil rod by forcing it to pass through a conical die. By this process, the steel can be strengthened by 100%.
- (b) **Concrete:** The requirement of concrete in CWPC is same as that of ordinary reinforced concrete.

9.4.17.2 Design

Similar to other reinforced concrete structures, CWPC structures have a complete set of design specification and computational approaches by which various members of the CWPC can be designed. In the design of prestressed members the function of pre-stressing force and pre-stressing losses should be calculated. CWPC members should be checked for its strength, stability and cracking resistance respectively at different stages including service, manufacturing, erection and construction. In designing members conformity to local specifications should be considered.

Cold drawn low carbon wire conforming to ASTM A615 or equivalent may be permitted for prestressing provided the mechanical requirements shown in Table 9.4.5 are satisfied.

Table 9.4.5 Tensile Strength and Elongation of Cold Drawn Wire

Diameter of wire (mm)	Minimum tensile strength (N/mm ²)	Minimum elongation (per cent)
3	650	2.0
4	600	2.5
5	550	3.0

9.4.18 External post-tensioning

- 9.4.18.1 Post-tensioning tendons shall be permitted to be external to any concrete section of a member. The strength and serviceability design methods of this Code shall be used in evaluating the effects of external tendon forces on the concrete structure.
- 9.4.18.2 External tendons shall be considered as unbonded tendons when computing flexural strength unless provisions are made to effectively bond the external tendons to the concrete section along its entire length.
- 9.4.18.3 External tendons shall be attached to the concrete member in a manner that maintains the desired eccentricity between the tendons and the concrete centroid throughout the full range of anticipated member deflection.
- 9.4.18.4 External tendons and tendon anchorage regions shall be protected against corrosion, and the details of the protection method shall be indicated on the drawings or in the project specifications.

9.4.19 Performance requirement of prestressed concrete design

9.4.19.1 Classification of performance requirement

After the outline of the member dimensions are determined and the most suitable kind and type of prestressing options are selected at the structural planning stage, the prestressed concrete noncomposite and composite structures and members shall satisfy all of the required performances such as safety, serviceability, restorability, durability, reparability, societal and environmental compatibility, etc. at every stage of design, construction and maintenance throughout the design life of the structure.

Table 9.4.8 gives the performance requirement of prestressed concrete structures and components and related performance items.

9.4.19.2 Performance verification method

- (a) Performance verification shall be based on the partial factor method on the basis of reliability theory and as a standard design procedure, it shall be based on the limit state method.
- (b) In general verification shall be based on design responses to design actions, design limits as determined by design material strengths, and individual partial factors. The performance of the structure shall, in general, be verified using Equations 9.4.19-1 and 9.4.19-2:

Table 9.4.8 Classification of performance requirement for prestressed concrete structures

Performance requirements	Performance item	Examples of check items	Example of verification index
Safety	Structural safety	Resistance of whole structure, components, stability, deformation performance	Stress resultant, stress
	Public safety	Injury to users and third parties	-
Serviceability	Live load operating performance	Soundness and rigidity of structures/members under usual conditions	Floor flatness, deformation of main girder
	User comfort	User-comfort under walking-induced vibrations	Natural frequency of main girders
Restorability	Restorability after earthquake, cyclone, tidal bore, fire, etc.	Level of damage (ease of restoration)	Response value (damage level)/limit value of performance (damage level)
Durability	Fatigue resistance	Fatigue durability against variable actions	Equivalent stress range/allowable stress range
	Corrosion resistance	Rust prevention and corrosion protection performance of steel material	Corrosion environment and surface finish, paint specification
	Resistance to material deterioration	Concrete deterioration	Water- cement ratio, cover of concrete
	Maintainability	Ease of maintenance (inspection, ease of repair, etc.) and ease of restoration	-
Social and	Social compatibility	Appropriateness of partial factor	Partial factor, structural factor,

Performance requirements	Performance item	Examples of check items	Example of verification index
environmental compatibility		(consideration of social importance of structure)	etc.
	Economic rationality	Social utility during life cycle of structure	Life cycle cost (LCC), life cycle utility (LTU)
	Environmental compatibility	Noise, vibration, environmental impact, aesthetics, etc.	Noise and vibration levels for surrounding residents, aesthetic reaction to structural shape and color, monumental aspect, etc.
Constructability / workability	Safety during construction	Safety during construction	Stress resultant, stress, deformation
	Initial soundness	Material quality, welding quality, etc.	Material properties, workmanship
	Ease of construction	Ease of fabrication and construction work	User-friendly construction methodology conceived at design stage

$$\gamma_i \frac{S_d}{R_d} \leq 1.0 \quad (9.14.19-1)$$

$$\gamma_i \frac{\sum \gamma_a \cdot S(\gamma_f \cdot F_k)}{R(f_k / \gamma_m)} \leq 1.0 \quad (9.14.19-2)$$

where, R_d : design resistance
 f_k : characteristic value of material strength
 γ_m : material factor
 γ_b : structural member factor
 $R(\dots)$: function for calculating limit value of structure from material strength
 S_d : design response
 F_k : individual characteristic value of action
 γ_a : structural analysis factor
 γ_f : action factor corresponding to each action (load factor)
 $S(\dots)$: function for calculating response value of structure from action
 γ_i : structural factor

- (a) During design, a verification shall be carried out for every limit state that can be considered.
(b) The flow chart explaining the concept of verification of safety is given in Fig.9.4.19.

Design resistance R

Characteristic value of material strength: f_k

γ_m ↓

Design value of material strength: $f_d = f_k / \gamma_m$

↓

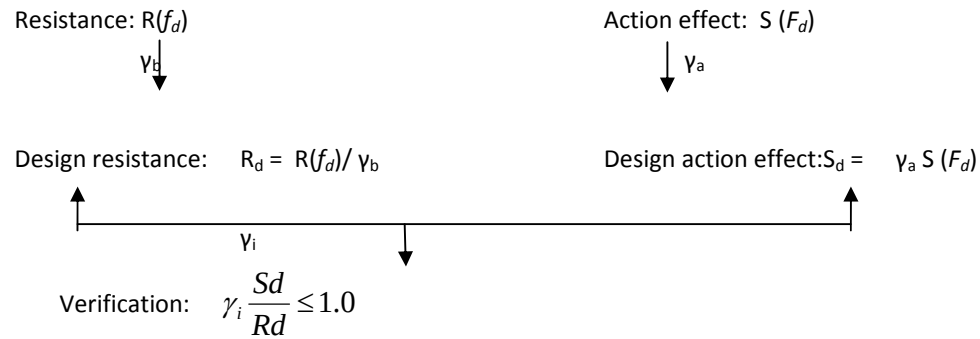
Design action effect

Characteristic value of action: F_k

↓ γ_f

Design value of action: $F_d = \gamma_f F_k$

↓



9.4.19.3 Partial factors

(a) Partial factors shall be determined on the concept given i. and ii. below.

- i) The material factor, structural member factor, structural analysis factor, and action factor shall be determined in consideration of (i) unfavorable deviations from characteristic values, (ii) uncertainties in computational accuracy, and (iii) discrepancies between design and practice with respect to actions or structures and materials.

Table 9.4.9 shows the standard values of partial factors.

- ii) The structural factor γ_i shall be determined according to structural importance and also the social and economical impact of the structure reaching its limit state.

Table 9.4.10 shows the standard values of structural factor γ_i for different performance items.

Table 9.4.9 Standard values of partial factors

Performance item	Action factor γ_f	Structural analysis factor γ_a	Material factor γ_m	Structural member Factor γ_b
Structural safety	1.0 ~ 1.6	1.0 ~ 1.1	1.0 ~ 1.05	1.0 ~ 1.3
Serviceability (user comfort)	1.0	1.0	1.0 ~ 1.05	1.0
Durability (fatigue resistance)	1.0 ~ 1.1	1.0	1.0	1.0 ~ 1.1

Table 9.4.10 Standard values of structural factors

Performance item	Structural factor γ_i
Structural safety	1.0 ~ 1.2
Serviceability (User comfort)	1.0
Durability (fatigue resistance)	1.0

Part B-Material and Construction

9.5 Material and Construction

9.5.1 Materials

9.5.1.1 Concrete ingredients and applicable ASTM standards

Table 9.5.1 shows the list of commonly applicable standards for cement, coarse and fine aggregates, admixtures and mixing water.

Table 9.5.1 Cement, coarse and fine aggregates, admixtures, water and applicable standards

Material	Designation of the Standard	Title of the Standard
Concrete	ASTM C39	Compression testing of cylindrical concrete specimens
Cement	BDS EN 197-1, issued April 2003	Part 1: Composition, specifications and conformity criteria for common cements
Fine & coarse aggregates	ASTM C136-06	Standard test method for sieve analysis of fine and coarse aggregates
		Standard test method for organic impurities in fine aggregates for concrete
	ASTM C40-04	Clay lumps and friable particles
	ASTM C142	Specific gravity and absorption of coarse aggregate
	ASTM C127	Specific gravity and absorption of fine aggregate
	ASTM C128	Degradation of small-size coarse aggregate by L.A. abrasion test
	ASTM C131	Unit weights and voids in aggregates
		Surface moisture in fine aggregate Soundness of aggregates by use of sodium sulfate or magnesium sulfate
	ASTM C29	
	ASTM C70	Soundness of aggregates by use of sodium sulfate or magnesium sulfate
		Alkali reactivity, potential of cement aggregate combinations
	ASTM C88	Potential alkali reactivity of aggregates (Mortar-bar method)
		Sand equivalent value of soils and fine aggregate
Admixtures		
	ASTM C494	Type A – Water reducing
		Type B – Retarding
		Type C – Accelerating
		Type D – Water reducing and retarding
		Type E – Water reducing and accelerating
		Type F – Water reducing, high range
		Type G – Water reducing, high range and retarding
		Type S – Specific performance admixture

Material	Designation of the Standard	Title of the Standard
Mixing Water	ASTM C 1602/ C1602M-06	Standard specification for mixing water used in the production of hydraulic cement concrete

9.5.1.2 Reinforcing steel and applicable standards

Table 9.5.2 shows the types of reinforcing steel with the ASTM and BDS Designation standard specifications.

Table 9.5.2 List of Standards for the reinforcing steel

Material	Designation of the Standard	Title of the Standard
Reinforcing Steel	ASTM A615/ A615M-04a	Standard specifications for deformed and plain carbon steel bars for concrete reinforcement
	ASTM A706/ A706M-04a	Standard specifications for low-alloy steel deformed and plain carbon steel bars for concrete reinforcement
	BDS ISO 6935-2: 2009/ ISO 6935-2: 2007	Bangladesh standard, Steel for the reinforcement of concrete, Part 2: Ribbed bars (1 st revision)

9.5.1.3 Prestressing steel and applicable ASTM standards

Table 9.5.3 shows the types of high tensile prestressing steel and cold drawn wires used for prestressing, with the ASTM Designation standard specifications.

Table 9.5.3 List of Standards for the prestressing steel

Material	Designation of the Standard	Title of the Standard
Prestressing Steel	ASTM A416-416M-02	Standard specification for steel, wire, hard drawn for prestressing concrete pipe
	ASTM A648-04a	Standard specification for uncoated high strength steel bars for prestressing concrete

9.5.2 Construction of prestressed concrete structures

9.5.2.1 Post-tensioning ducts

9.5.2.1.1 Corrosion protection for unbonded tendons

- Unbonded prestressing steel shall be encased with sheathing. The prestressing steel shall be completely coated and the sheathing around the prestressing steel filled with suitable material to inhibit corrosion.
- Sheathing shall be watertight and continuous over entire length to be unbonded.
- For applications in corrosive environments, the sheathing shall be connected to all stressing, inter-mediate and fixed anchorages in a watertight fashion.
- Unbonded single-strand tendons shall be protected against corrosion in accordance with ACI 423.7.

9.5.2.2 Post-tensioning ducts

- (a) Ducts for grouted tendons shall be mortar-tight and nonreactive with concrete, prestressing steel, grout, and corrosion inhibitor.
- (b) Ducts for grouted single-wire, single-strand, or single-bar tendons shall have an inside diameter at least 6 mm larger than the prestressing steel diameter.
- (c) Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area at least two times the cross-sectional area of the prestressing steel.
- (d) Ducts shall be maintained free of ponded water if members to be grouted are exposed to temperatures below freezing prior to grouting.

9.5.2.3 Grout for bonded tendons

9.5.2.3.1 Materials for grouts

- (a) Grout shall consist of Portland cement and water; or Portland cement, sand, and water.
- (b) Materials for grout shall conform to 9.5.2.2.1c., d., e. and f. below.
- (c) Portland cement shall conform to 9.5.1.1.
- (d) Water shall conform to 9.5.1.1.
- (e) Sand, if used, shall conform to 9.5.1.1 except that gradation shall be permitted to be modified as necessary to obtain satisfactory workability.
- (f) Admixtures conforming to 9.5.1.1 and known to have no injurious effects on grout, steel, or concrete shall be permitted. Calcium chloride shall not be used.

9.5.2.3.2 Selection of grout proportions

Proportions of materials for grout shall be based on either a. or b. below.

- (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.
- (c) Cement used in the Work shall correspond to that on which selection of grout proportions was based.
- (d) Water content shall be minimum necessary for proper pumping of grout; however, water-cement ratio shall not exceed 0.45 by weight.
- (e) Water shall not be added to increase grout flowability that has been decreased by delayed use of the grout.

9.5.2.3.3 Mixing and pumping of grout

- (a) 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 the ducts.
- (b) Temperature of members at time of grouting shall be above 2°C and shall be maintained above 2°C until field-cured 50 mm cubes of grout reach a minimum compressive strength of 5.5 N/mm².
- (c) Grout temperatures shall not be above 32°C during mixing and pumping.

9.5.2.4 Protection for prestressing steel during welding

Burning or welding operations in the vicinity of prestressing steel shall be performed so that prestressing steel is not subject to excessive temperatures, welding sparks, or ground currents.

9.5.2.5 Application and measurement of prestressing force

9.5.2.5.1 Prestressing force shall be determined by both of (a) and (b):

- (a) Measurement of steel elongation. Required elongation shall be determined from average load-elongation curves for the prestressing steel used;

- (b) Observation of jacking force on a calibrated gage or load cell or by use of a calibrated dynamometer.
- 9.5.2.5.2 Cause of any difference in force determination between 9.5.2.4.1 (a) and (b) that exceeds 5 percent for pretensioned elements or 7 percent for post-tensioned construction shall be ascertained and corrected.
- 9.5.2.5.3 Where the transfer of force from the bulk- heads of pretensioning bed to the concrete is accomplished by flame cutting prestressing steel, cutting points and cutting sequence shall be predetermined to avoid undesired temporary stresses.
- 9.5.2.5.4 Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to concrete.
- 9.5.2.5.5 Total loss of prestress due to unreplaced broken prestressing steel shall not exceed 2 percent of total prestress.
- 9.5.2.6 **Post-tensioning anchorages and couplers**
- 9.5.2.6.1 Anchorages and couplers for bonded and unbonded tendons shall develop at least 95 percent of the f_{pu} when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100 percent of f_{pu} shall be developed at critical sections after the prestressing steel is bonded in the member.
- 9.5.2.6.2 Couplers shall be placed in areas approved by the licensed design professional and enclosed in housing long enough to permit necessary movements.
- 9.5.2.6.3 In unbonded construction subject to repetitive loads, attention shall be given to the possibility of fatigue in anchorages and couplers.
- 9.5.2.6.4 Anchorages, couplers, and end fittings shall be permanently protected against corrosion.

9.5.3 Performance requirement of material

9.5.3.1 Performance requirement of material

- The fundamental performance requirement of materials forming the structure is that they should be able to resist actions such as the various loadings to which the structure is exposed.
- Materials forming the structure should not reach unexpected limit states as a result of deterioration phenomena during the working life of the structure.
- Materials-related energy consumption and CO₂ discharges should be minimized, while recyclability should be high.
- Any materials that escape into the surrounding environment during construction and service should not have a strong impact on human beings, animals and plants.

Commentary:

Corresponding to design requirements, the materials should be evaluated to ensure that their properties are suitable with respect to strength (tensile, compressive and shear), deformation (e.g. elastic modulus), heat resistance and water tightness.

The characteristic values obtained from the tests, complying appropriate BDS, ASTM, BS, or equivalent standards, on such specimens should be converted to suit the design calculation models using appropriate conversion factors or functions.

The characteristic value of material strength f_k is calculated from test results using Eq. 9.5.3-1.

$$f_k = f_m - k\sigma \quad (9.5.3-1)$$

where f_m : mean of test values, σ : standard deviation of test values, and k : coefficient of variance. The coefficient k is determined from the probability of obtaining a test value less than the characteristic value and the probability distribution of test results. The 5% fractile value is often taken as the characteristic value. In this case, the value of k is 1.64 if the normal distribution is assumed for the test values.

At the structural design stage, verification shall be performed so that response value is less than or equal to the limit value of performance throughout both construction period and working life. At the end of construction stage, just completed structure shall fulfill the all required performances considered in its design.

Part C-Maintenance

9.6 Maintenance

9.6.1 General

If the prestressed concrete structure is designed and constructed in accordance with the appropriate concepts described in Part I and II of this Chapter, based on which the durability is checked by verifying the performance requirements of the concrete and its constituent materials, it is not likely that structural deterioration would become so significant as to degrade the performance of the structure. On the other hand it is not easy to estimate the performance degradation process of the structure during its service life accurately. Also, it is difficult to completely avoid construction defects at all construction stages. Therefore, the new structure should be appropriately maintained by routine and regular inspections, based on an adequate maintenance plan formulated at the design stage.

For existing structures, deterioration may be evident in some cases, with the performance having been degraded. The defects of such structures should be accurately assessed and identified as initial defects, damage, or deteriorations. Major causes for such defects should be identified subsequently so that appropriate remedial actions can be selected. The initial defects and damage should be treated promptly and appropriately including emergency treatments. When the deterioration that would degrade the performance is evident, the deterioration mechanisms should be identified and appropriate maintenance, carried out based on the results of deterioration prediction and performance degradation evaluation.

9.6.2 Classification of Maintenance Action

Maintenance actions shall be classified into different categories depending on such factors as the importance of the structure, design life, impact on a third party, environmental conditions, ease of maintenance, and cost.

In the view of the above, four categories are recommended for the classifications of the maintenance actions:

Category A – Preventive maintenance

Maintenance to prevent deterioration which would otherwise lead to unsatisfactory structural performance.

Category A structures are those

- for which remedial actions are difficult to take after deterioration becomes apparent;
- of which deterioration must not be apparent;
- having a long design life.

Structures in this category generally have a high degree of importance which in many cases require monitoring.

Category B – Corrective maintenance

Maintenance to restore the performance level and/or to reduce the rate of deterioration so as to maintain satisfactory structural performance.

Category B structures are those for which

- remedial measures can be taken after deterioration becomes apparent;
- apparent deterioration causes no appreciable inconvenience.

Category C – Observational maintenance

Maintenance in which visual inspection is necessary without any remedial action regardless of the deterioration level.

Category C structures are those

- for use as long as they are usable;
- for which ensuring safety from threats posed to third parties is the only requirement.

Category D – Indirect maintenance

Maintenance in which no direct inspection is necessary or possible.

Category D structures are those for which direct inspection is extremely difficult. For these reasons, non-inspection maintenance after the initial inspection is carried out not as routine or regular inspection, but as extraordinary inspection following natural disasters, accidents, etc.

9.6.3 Maintenance Record

Records, drawings and related documents prepared during the time of planning, design and construction shall be referred to and made use of while developing an appropriate methodology for maintenance covering inspection and repairs.

Commentary:

A thorough study of the planning, design and construction related documents often provide insights into the inherent weaknesses of the structure which in turn often serve as pointers for further detailed inspection and/or repairs.

Furthermore, a clear record should be kept of the difficulties encountered, remedial actions taken and any deviation from the design drawings. These record also serve as a valuable reference in the design and construction of similar structures and their subsequent inspections.

9.6.4 Inspection

9.6.4.1 General

On the basis of the methods used in the frequency and timing, inspection shall be classified as initial inspection, routine inspection, regular inspection, detailed inspection, extraordinary inspection, and monitoring.

9.6.4.2 Initial inspection

Initial inspection is intended to examine whether the structure is adequately constructed. It also allows the collection of basic data for initiating a maintenance program. Initial inspection shall also be carried out just after the completion of remedial actions.

Initial inspection should cover the external appearance of the structure, variation of concrete quality, existence of construction defects, construction errors on reinforcing and prestressing bar arrangement, and so on.

9.6.4.3 Routine inspections

It shall be carried out on a routine basis at certain intervals without making any specific effort to identify signs of deterioration, if any, and the time of their first appearance. The exact tools to be used and the frequency of such inspections may be decided on the basis of such factors as the likely mechanisms of such deterioration, environmental conditions, importance of the structure, and the maintenance action classification.

A routine inspection should cover the external appearance of the structure including cracks, spalling, delamination, color changes, rust stain from reinforcement, and isolation of free lime from concrete.

9.6.4.4 Regular inspection

It shall be carried out at regular intervals using appropriate tools to identify signs of deterioration and the time of their first appearance. Efforts shall be made during a regular inspection to observe the structure closely to obtain details which will be difficult to gather during a routine inspection.

Visual inspection and/or hammering inspection are carried out mainly to obtain more details on the items inspected in a routine inspection. In addition, inspections by using appropriate non-destructive tests or taking concrete cores etc. can be effectively combined with the visual inspection.

9.6.4.5 Detailed inspection

Detailed inspection shall be done when

- (a) some signs of deterioration or a change in the performance level are observed during a routine and/or regular inspection;
- (b) it is difficult to obtain reliable and accurate information during a routine and/or regular inspection;

- (c) it is found that the structural integrity of the structure has been adversely affected by the extent of the deterioration;
- (d) more detailed information is required before deciding on the necessity and scope for undertaking a major repair, rehabilitation or strengthening work.

9.6.4.6 Extraordinary inspection

It shall be carried out after a structure has been subjected to an accidental load to assess the extent of the damage and the need for remedial actions. Such accidental loads may include those caused by an earthquake, storm, flood, fire, explosion, etc.

9.6.5 Monitoring

The deterioration and/or performance of the concerned structure as determined in 9.6.2, shall be monitored, through continuous recording of the appropriate data, together with routine and regular inspections, so that the appropriate remedial actions can be taken before the deterioration becomes detrimental to the appearance and other performance of the structure.

9.6.6 Deterioration Mechanism and Prediction

9.6.6.1 General

The prevailing state of the concerned structure shall be evaluated as properly as possible according to the inspection results, design and construction records, environmental conditions, and any other relevant information. Then when any deterioration is found, the possible causes of the deterioration and the corresponding mechanism can be appropriately estimated.

9.6.6.2 Identification of deterioration mechanisms

Deterioration of a structure is caused by the environmental actions and loading conditions. Environment-oriented deterioration includes carbonation-induced deterioration, chloride-induced deterioration, chemical attack, alkali-aggregate reaction, etc. On the other hand external force-oriented deterioration includes fatigue, excessive loading, and differential settlement of the support.

9.6.6.3 Deterioration factors

Deterioration factors may be classified into those

- (a) external to structures such as temperature, humidity and any other environmental characteristics; and
- (b) internal to the structure such as design parameters and quality control during construction.

Commentary:

Design factors include the geometry of the members/ segments, crack width specifications, concrete cover to reinforcing bar and prestressing steel/ducts, and design strength. Construction factors include material selection, mix proportions, transportation, placement, and curing methods.

9.6.6.4 Determination of deterioration levels and rates

The level of deterioration and/or performance shall be determined based on the results of inspections and simulations using appropriate models for the mechanisms of deterioration.

The following features appearing on the surface of the structure may be used for evaluating the degree of deterioration and the level of performance:

- (a) crack pattern, length and width;
- (b) the extent of delamination, peeling and spalling of concrete cover, and scaling and degradation areas;
- (c) abnormal hammer tapping sound and the extent of abnormality;
- (d) presence and degree of exudation of rust and efflorescence and water leakage.

9.6.7 Evaluation and Decision Making

9.6.7.1 General

In general, the deterioration and performance degradation of a structure progress monotonically. The decision, therefore, should be made based on the evaluation outcome of the performance of the structure at the time of inspection and at the end of its design life.

9.6.7.2 Threshold level

The threshold level of the structure's degraded performance shall be specified in accordance with the requirements of safety, functionality, appearance, societal friendliness and such other factors, taking into consideration the type, importance and maintenance level of the structure and the environmental conditions.

9.6.7.3 Evaluation of inspection results

The results from routine and regular inspections shall be evaluated and a decision shall be made whether a detailed inspection is required or otherwise.

The results from the detailed and/or extraordinary inspections shall be evaluated and a decision shall be made whether a remedial action is required or otherwise.

Immediate remedial actions shall be taken in cases where deterioration, damage and/or initial defects are found to be hazardous to third parties.

9.6.8 Remedial Action

9.6.8.1 General

A remedial action on a deteriorated structure shall be taken on the basis of the inspection results, importance of the structure, maintenance classification, and the threshold level of deterioration and/or performance.

Commentary:

Repair and strengthening are the main techniques of remedial actions of which details are described in 9.6.8.3 and 9.6.8.4 respectively. The following measures are also included in the remedial actions.

Intensified inspection: inspection may be carried out by suitably increasing one or more of the following: frequency of inspection, number of inspection items, and the locations for inspection.

Usage restriction: suitable restriction shall be imposed on the maximum live load that the structure may carry, depending on the level of deterioration observed.

Functional improvement or restoration: this may include an appearance improvement that beautifies a structure with suitably painting or placing additional concrete, and so on.

Dismantling and removal: in a case when the deterioration of a structure is too severe for its structural performance to be sufficiently restored, and dismantling or the removal is one of the choices as the remedial measures.

Special care for emergency: when a deteriorated structure poses an immediate threat to the environment, its users, or third parties, suitable emergency action shall be taken immediately.

9.6.8.2 Selection of remedial action

Selection of methods and materials suitable for the relevant deterioration mechanism and degree of performance degradation is particularly important for measures for which wide varieties of methods and materials are available. Care should be taken as the method of restoring the performance may vary depending on the deterioration mechanism, even if the level of performance is the same.

9.6.8.3 Repair

9.6.8.3.1 General

Repair of a structure refers to the remedial action taken to prevent or slow down its further deterioration and reduce the possibility of damage to its users or third parties.

Types of repair include (i) repair of defects such as cracking and peeling; (ii) removal of concrete damaged by deterioration due to carbonation and such like; (iii) surface coating to prevent re-intrusion of hazardous substances.

9.6.8.3.2 Preparation and execution

A complete plan for the repair work including methods of repair, materials to be used, and tests to ensure the quality of work, shall be developed before the repair work commences.

Repair works shall be carried out with minimum disturbances to the surrounding environment. Necessary tests to ensure the quality of the repair work shall be carried out. Detailed record of the repair work shall be maintained for future reference.

9.6.8.3.3 Methods and materials

Some current repair methods and associated materials are

- crack repair by injecting epoxy;
- section repair including patching using polymer cement mortar;
- surface protection by resin or mortar;
- cathodic protection;
- re-alkalization;
- desalination wherever required.

Commentary:

Development of a repair plan comprises the selection of a repair method suitable for the deterioration mechanism, establishment of the required repair level, and decisions on the repair policy, specifications for the repair materials, sectional dimensions after repair, and execution methods.

9.6.8.4 Strengthening

9.6.8.4.1 General

Strengthening of a structure refers to the remedial action taken to restore or improve its structural properties including load carrying capacity and stiffness, to a level which is equal to or higher than that of the original design.

Commentary:

Strengthening methods include (i) replacement of members; (ii) an increase in the cross-sectional area of concrete; (iii) addition of members; (iv) an increase of the support points; (v) addition of strengthening members; (vi) external prestressing, etc.

9.6.8.4.2 Preparation and execution

Strengthening of a structure shall be preceded by a thorough investigation of its deterioration considering such factors as the remaining design life, deterioration mechanism, possible causes and extent of deterioration, the remaining and desired load-carrying capacity or stiffness, importance of the structure, maintenance classification, and any remedial actions taken previously.

A complete plan for the strengthening work including design calculations, methods of strengthening, materials to be used, and tests to ensure quality of the work, shall be developed before work commences.

Strengthening work shall be carried out with minimum disturbance to the surrounding environment and the service condition of the structure.

9.6.8.4.3 Methods and materials

Some current methods and associated materials for strengthening are

- external bonding viz plate or sheet bonding and over or under-laying using steel or carbon sheets;

- external prestressing using additional tension cables;
- addition of girders, braces and/or supports;
- replacement of members;
- seismic isolation.

Commentary:

When selecting a strengthening method, it is necessary to consider effects of strengthening, constructability, cost-effectiveness, and impact on the community/environment during execution. It is also important to consider the ease of maintenance after strengthening and any influence on the landscape.

9.6.8.5 **Record**

9.6.8.5.1 **General**

Records shall be kept and preserved for future reference. Such records shall include details concerning the design, inspection and evaluation procedures, plans and execution of any repair and/or strengthening work undertaken, and other such information.

9.6.8.5.2 **Preservation**

The maintenance records of a structure shall be preserved while the structure remains in service. It is also desirable that such records be preserved for an indefinite period as a useful reference for the construction and maintenance of other similar structures.

Commentary:

It is important to devise a format that makes it easy to understand the history of a structure by simply referring to records. The records should be made accessible at all times.

9.6.8.5.3 **Method and item of recording**

Records shall be kept in an easy-to-understand format.

The items to be recorded shall include references to concerned agencies, drawings, immediate and nearby environment, classification of structure, results of deterioration rate estimation, results of any inspections carried out, evaluation of the structure, and details of the plan and actual execution of remedial and other actions.

Chapter 10

STEEL STRUCTURES

10.1 General Provisions for Structural Steel Buildings and Structures

This section states the scope of the Specification, summarizes referenced specification, code, and standard documents, and provides requirements for materials and contract documents.

10.1.1 Scope

The specification contained in chapter 10 of Part 6 of this code sets forth criteria for the design, fabrication, and erection of *structural steel* buildings and other structures, where other steel-structures are defined as those structures designed, fabricated, and erected in a manner similar to steel-buildings, with building-like vertical and lateral load resisting elements. Where conditions are not covered by this specification, designs are permitted to be based on tests or analysis, subject to the approval of the *authority having jurisdiction*. Alternate methods of analysis and design shall be permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

10.1.1.1 Low-Seismic Applications: When the seismic response modification coefficient, R , (as specified in Chapter 2 of Part 6) is taken equal to or less than 3, the design, fabrication, and erection of structural-steel-framed buildings and other steel-structures shall comply with this specification except that such structures need not to comply with the specifications set forth in *Section 10.20 Seismic Provisions*.

10.1.1.2 High-Seismic Applications: When the seismic response modification coefficient, R , (as specified in Chapter 2 of Part 6) is taken greater than 3, the design, fabrication and erection of structural-steel-framed buildings and other structures shall comply with the requirements in the *Section 10.20 Seismic Provisions*, in addition to the provisions of other sections (whichever applicable) this specification.

10.1.2 Referenced Specifications, Codes and Standards

The following specifications, codes and standards are referenced in this Specification:

ACI International (ACI)

ACI 318-02 *Building Code Requirements for Structural Concrete and Commentary*

ACI 318M-02 *Metric Building Code Requirements for Structural Concrete and Commentary*

American Institute of Steel Construction, Inc. (AISC)

AISC 303-05 *Code of Standard Practice for Steel Buildings and Bridges*

ANSI/AISC 341-05 *Seismic Provisions for Structural Steel Buildings*

ANSI/AISC N690-1994(R2004) *Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement No. 2*

ANSI/AISC N690L-03 *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities*

American Society of Civil Engineers (ASCE)

SEI/ASCE 7-02 *Minimum Design Loads for Buildings and Other Structures*

ASCE/SFPE 29-99 *Standard Calculation Methods for Structural Fire Protection*

American Society of Mechanical Engineers (ASME)

ASME B18.2.6-96 *Fasteners for Use in Structural Applications*

ASME B46.1-95 *Surface Texture, Surface Roughness, Waviness, and Lay*

American Iron and Steel Institute (AISI)

North American Specification for the Design of Cold Formed Steel Structural Members (AISI/COS/NASPEC 2001).
Code of Standard Practice for Cold-Formed Steel Structural Framing, 2005

ASTM International (ASTM)

A6/A6M-04a *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A36/A36M-04 *Standard Specification for Carbon Structural Steel*

A53/A53M-02 *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*

A193/A193M-04a *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service*

A194/A194M-04 *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

A216/A216M-93(2003) *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High Temperature Service*

A242/A242M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel*

A283/A283M-03 *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

A307-03 *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

A325-04 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

A325M-04 *Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)*

A354-03a *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

A370-03a *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A449-04 *Standard Specification for Quenched and Tempered Steel Bolts and Studs*

A490-04 *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

A490M-04 *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*

A500-03a *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

A501-01 *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*

A502-03 *Standard Specification for Steel Structural Rivets*

A514/A514M-00a *Standard Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*

A529/A529M-04 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

A563-04 *Standard Specification for Carbon and Alloy Steel Nuts*

A563M-03 *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

A568/A568M-03 *Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-04 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick*

A606-04 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A618/A618M-04 *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*

A673/A673M-04 *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*

A668/A668M-04 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*

A709/A709M-04 *Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges*

A751-01 *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*

A847-99a(2003) *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*

A852/A852M-03 *Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 485 MPa Minimum Yield Strength to 100 mm Thick*

A913/A913M-04 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*

A992/A992M-04 *Standard Specification for Structural Steel Shapes*

A1011/A1011M-04 *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability*

C33-03 *Standard Specification for Concrete Aggregates*

C330-04 *Standard Specification for Lightweight Aggregates for Structural Concrete*

E119-00a *Standard Test Methods for Fire Tests of Building Construction and Materials*

E709-01 *Standard Guide for Magnetic Particle Examination*

F436-03 *Standard Specification for Hardened Steel Washers*

F959-02 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

F1554-99 *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

F1852-04 *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

American Welding Society (AWS)

AWS D1.1/D1.1M-2004 *Structural Welding Code—Steel*

AWS A5.1-2004 *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.5-96 *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.17/A5.17M-97 *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.18:2001 *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.20-95 *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding*

AWS A5.23/A5.23M-97 *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.25/A5.25M-97 *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*

AWS A5.26/A5.26M-97 *Specification for Carbon and Low-Alloy Steel Electrodes for Electrode Gas Welding*

AWS A5.28-96 *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.29:1998 *Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding*

Research Council on Structural Connections (RCSC)

Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2004

Bangladesh Standards and Testing Institute (Bangladesh Standards or BDS)

10.1.3 Material

10.1.3.1 Structural Steel Materials

- a) Regular Structural Steel: Material test reports from an acceptable testing laboratory shall constitute sufficient evidence of conformity with one of the above listed ASTM standards. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for tubing and pipe, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those

product forms. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

Structural steel material conforming to one of the following specifications is approved for use under this Specification:

(i) Hot-rolled structural shapes

ASTM A36/A36M, ASTM A529/A529M, ASTM A572/A572M, ASTM A588/A588M, ASTM A709/A709M, ASTM A913/A913M, ASTM A992/ A992M

(ii) Structural tubing

ASTM A500, ASTM A501, ASTM A618, ASTM A847, BDS 1031:2006

(iii) Pipe

ASTM A53/A53M, Gr. B, BDS 1031:2006

(iv) Plates

ASTM A36/A36M, ASTM A242/A242M, ASTM A283/A283M, ASTM A514/A514M, ASTM A529/A529M, ASTM A572/A572M, ASTM A588/A588M, ASTM A709/A709M, ASTM A852/A852M, ASTM A1011/A1011M, BDS 1122:1987 Reaffirmed 2007

(v) Bars

ASTM A36/A36M, ASTM A529/A529M, ASTM A572/A572M, ASTM A709/A709M, BDS ISO 6935-1:2006, BDS ISO 6935-2:2006

(vi) Sheets

ASTM A606, A1011/A1011M SS, HSLAS, AND HSLAS-F, BDS 1122:1987 Reaffirmed 2007

- b) Unidentified Steel: Unidentified steel free of injurious defects is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

- c) Rolled Heavy Shapes: ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm, used as members subject to primary (computed) tensile *forces* due to tension or flexure and spliced using *complete-joint-penetration groove welds* that fuse through the thickness of the member, shall be specified as follows. The contract documents shall require that such shapes be supplied with *Charpy V-Notch (CVN) impact test* results in accordance with ASTM A6/A6M, Supplementary Requirement S30, *Charpy V-Notch Impact Test for Structural Shapes – Alternate Core Location*. The impact test shall meet a minimum average value of 27 J absorbed energy at +21⁰ C.

The above requirements do not apply if the splices and connections are made by bolting. The above requirements do not apply to hot-rolled shapes with a flange thickness exceeding 50 mm that have shapes with flange or web elements less than 50 mm thick welded with complete-joint-penetration groove welds to the face of the shapes with thicker elements.

- d) Built-Up Heavy Shapes: Built-up cross-sections consisting of plates with a thickness exceeding 50 mm, used as members subject to primary (computed) tensile *forces* due to tension or flexure and spliced or connected to other members using *complete-joint- penetration groove welds* that fuse through the thickness of the plates, shall be specified as follows. The contract documents shall require that the steel be supplied with *Charpy V-Notch impact test results* in accordance with ASTM A6/A6M, Supplementary Requirement S5, *Charpy V-Notch Impact Test*. The impact test shall be conducted in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 27 J absorbed energy at +21⁰ C.

The above requirements also apply to built-up cross-sections consisting of plates exceeding 50 mm that are welded with complete-joint-penetration groove welds to the face of other sections.

- e) Cold Form Sections: Specifications for cold form shapes regarding their use as structural members is not covered in Section 10. For such type of structural steel, AISI standard (AISI/COS/NASPEC 2001) or equivalent may be followed.

10.1.3.2 **Steel Castings and Forgings:** Cast steel shall conform to ASTM A216/A216M, Gr. WCB with Supplementary Requirement S11. Steel forgings shall conform to ASTM A668/A668M. Test reports

produced in accordance with the above reference standards shall constitute sufficient evidence of conformity with such standards.

- 10.1.3.3 Bolts, Washers and Nuts: Bolt, washer, and nut material conforming to one of the following ASTM specifications is approved for use under this Specification:

(1) **Bolts:** ASTM A307, ASTM A325, ASTM A325M, ASTM A449, ASTM A490, ASTM A490M, ASTM F1852

(2) **Nuts:** ASTM A194/A194M, ASTM A563, ASTM A563M

(3) **Washers:** ASTM F436, ASTM F436M

(4) **Compressible-Washer-Type Direct Tension Indicators:** ASTM F959, ASTM F959M

- 10.1.3.4 **Anchor rods and Threaded rods:** Anchor rod and threaded rod material conforming to one of the following ASTM specifications is approved for use under this Specification:

ASTM A36/A36M, ASTM A193/A193M, ASTM A354, ASTM A449, ASTM A572/A572M, ASTM A588/A588M, ASTM F1554

A449 material is acceptable for high-strength anchor rods and threaded rods of any diameter.

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

- 10.1.3.5 **Filler metal and Flux for Welding:** Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society: AWS A5.1, AWS A5.5, AWS A5.17/A5.17M, AWS A5.18, AWS A5.20, AWS A5.23/A5.23M, AWS A5.25/A5.25M, AWS A5.26/A5.26M, AWS A5.28, AWS A5.29, AWS A5.32/A5.32M

- 10.1.3.6 **Stud Shear Connectors:** Steel stud *shear connectors* shall conform to the requirements of *Structural Welding Code—Steel*, AWS D1.1.

10.1.4 Structural Design Drawings and Specifications

The design drawings and specifications shall meet the requirements specified in this specification (Sections 10.1 through 10.20) and shall be prepared and presented in an internationally approved standard in accordance with the provisions of Section 10.13, except for deviations specifically identified in the design drawings and/or specifications and approved by an appropriate authority.

10.2 General Design Requirements

The general requirements for the analysis and design of steel buildings and structures that are applicable to all sections of Chapter 10 of Part 6 are given in this section.

10.2.1 General Provisions

The design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis. Unless restricted by the applicable code, lateral load resistance and stability may be provided by any combination of members and connections.

10.2.2 Loads and Load Combinations

The loads and load combinations shall be as stipulated in Chapter 2 of Part 6 of this code. For design purposes, the nominal loads shall be taken as the loads stipulated in the Chapter 2

10.2.3 Design Basis

Designs shall be made according to the provisions for Load and Resistance Factor Design (LRFD) or to the provisions for Allowable Strength Design (ASD).

- 10.2.3.1 **Required Strength:** The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated in Chapter 2 of Part 6. Design by elastic, inelastic or plastic analysis is permitted. Provisions for inelastic and plastic analysis

are as stipulated in Section 10.15, Inelastic Analysis and Design. The provisions for moment redistribution in continuous beams in Section 10.15.3 are permitted for elastic analysis only.

10.2.3.2 **Limit States:** Design shall be based on the principle that no applicable strength or serviceability limit state shall be exceeded when the structure is subjected to all appropriate load combinations.

10.2.3.3 **Design for Strength Using Load and Resistance Factor Design (LRFD):** Design according to the provisions for Load and Resistance Factor Design (LRFD) satisfies the requirements of this Specification when the design strength of each structural component equals or exceeds the required strength determined on the basis of the LRFD load combinations as specified in Chapter 2 of Part 6. All provisions of this Specification, except for those in Section 10.2.3.4, shall apply.

Design shall be performed in accordance with Equation 10.2.1:

$$\leq \quad (10.2.1)$$

where

R_u = required strength (LRFD)

R_n = nominal strength, specified in Sections 10.2 through 10.20

ϕ = resistance factor, specified in Sections 10.2 through 10.20

ϕR_n = design strength

10.2.3.4 **Design for Strength Using Allowable Strength Design (ASD):** Design according to the provisions for Allowable Strength Design (ASD) satisfies the requirements of this Specification when the allowable strength of each structural component equals or exceeds the required strength determined on the basis of the ASD load combinations as specified in Chapter 2 of Part 6. All provisions of this Specification, except those of Section 10.2.3.3, shall apply.

Design shall be performed in accordance with Equation 10.2.2:

$$\leq / \Omega \quad (10.2.2)$$

where

R_a = required strength (ASD)

R_n = nominal strength, specified in Sections 10.2 through 10.20

Ω = safety factor, specified in Sections 10.2 through 10.20

R_n / Ω = allowable strength

10.2.3.5 **Design for Stability:** Stability of the structure and its elements shall be determined in accordance with Section 10.3.

10.2.3.6 **Design for Connection:** Connection elements shall be designed in accordance with the provisions of Section 10.10 and 10.11. The forces and deformations used in design shall be consistent with the intended performance of the connection and the assumptions used in the structural analysis.

10.2.3.6.1 **Simple Connection:** A simple connection transmits a negligible moment across the connection. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. Inelastic rotation of the connection is permitted.

10.2.3.6.2 **Moment Connection:** A moment connection transmits moment across the connection. Two types of moment connections, FR and PR, are permitted, as specified below.

a) **Fully-Restrained (FR) Moment Connections:** A fully-restrained (FR) moment connection transfers moment with a negligible relative rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states.

b) **Partially-Restrained (PR) Moment Connections:** Partially-restrained (PR) moment connections transfer moments, but the relative rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection shall

be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness, and deformation capacity at the strength limit states.

- 10.2.3.7 Design for Connection: The overall structure and the individual members, connections and connectors shall be checked for serviceability. Performance requirements for serviceability design are given in Section 10.12.
- 10.2.3.8 Design for Ponding: The roof system shall be investigated through structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with a slope of 20 mm per meter or greater toward points of free drainage or an adequate system of drainage is provided to prevent the accumulation of water. Methods of checking ponding are given in section 10.16.
- 10.2.3.9 Design for Fatigue: Fatigue shall be considered in accordance with Section 10.17, Design for Fatigue, for members and their connections subject to repeated loading. Fatigue need not be considered for seismic effects or for the effects of wind loading on normal building lateral load resisting systems and building enclosure components.
- 10.2.3.10 Design for Fire Conditions: Two methods of design for fire conditions are provided in Section 10.18, Structural Design for Fire Conditions: Qualification Testing and Engineering Analysis. Compliance with the fire protection requirements in Part 4 of this code shall be required in addition to satisfying the requirements of Section 10.18.
- 10.2.3.11 Design for Corrosion Effects: Where corrosion may impair the strength or serviceability of a structure, structural components shall be designed to tolerate corrosion or shall be protected against corrosion.
- 10.2.3.12 Design Wall Thickness for HSS: The design wall thickness, t , shall be used in calculations involving the wall thickness of hollow structural sections (HSS). The design wall thickness, t , shall be taken equal to 0.93 times the nominal wall thickness for electric-resistance-welded (ERW) HSS and equal to the nominal thickness for submerged-arc-welded (SAW) HSS.

10.2.3.13 Gross and Net Area Determination

10.2.3.13.1 Gross Area: The gross area, A_g , of a member is the total cross-sectional area.

10.2.3.13.2 Net Area: The net area, A_n of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as 2 mm greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section 10.10.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/(4g)$

where

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm.

g = transverse center-to-center spacing (gage) between fastener gage lines, mm.

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted HSS welded to a gusset plate, the net area, A_n , is the gross area the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

10.2.4 Classification of Sections for Local Buckling

Sections are classified as compact, noncompact, or slender-element sections. For a section to qualify as compact its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table 10.2. 1. If the width-thickness ratio of one or more compression elements exceeds λ_p , but does not exceed λ_r from Table 10.2.1, the section is noncompact. If the width-thickness ratio of any element exceeds λ_r , the section is referred to as a slender-element section.

10.2.4.1 Unstiffened Elements: For *unstiffened elements* supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

- (a) For flanges of I-shaped members and tees, the width b is one-half the full-flange width, b_f .
- (b) For legs of angles and flanges of channels and zees, the width b is the full *nominal dimension*.
- (c) For plates, the width b is the distance from the free edge to the first row of *fasteners* or line of welds.
- (d) For stems of tees, d is taken as the full nominal depth of the section.

10.2.4.2 Stiffened Elements: For *stiffened elements* supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

- a) For webs of rolled or *formed sections*, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.
- b) For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.
- c) For flange or *diaphragm plates* in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
- d) For flanges of rectangular hollow structural sections (HSS), the width b is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t , shall be taken as the *design wall thickness*, per Section 10.2.3.12.
- e) For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

10.2.5 Fabrication, Erection and Quality

Shop drawings, fabrication, shop painting, erection, and *quality control* shall meet the requirements stipulated in Section 10.13, Fabrication, Erection, and Quality Control.

10.3 Stability Analysis and Design

This section addresses general requirements for the stability analysis and design of members and frames of steel buildings and structures.

10.3.1 Stability Design Requirements

General Requirements: Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of *second-order effects* (including $P-\Delta$ and $P-\delta$ effects), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted. The methods prescribed in this Section and Section 10.14: *Direct Analysis Method*, satisfy these requirements. All component and *connection* deformations that contribute to the lateral displacements shall be considered in the stability analysis.

TABLE 10.2.1
Limiting Width-Thickness Ratios for
Compression Elements

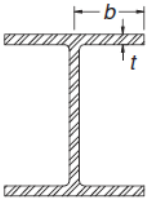
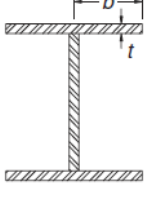
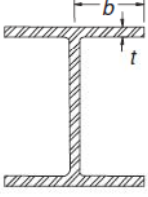
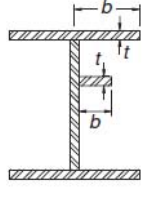
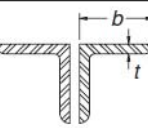
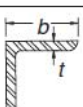
	Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios		Example
				λ_p (compact)	λ_r (noncompact)	
Unstiffened Elements	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_c E/F_y}^{[a],[b]}$	
	3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels	b/t	NA	$0.56\sqrt{E/F_y}$	
	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64\sqrt{k_c E/F_y}^{[a]}$	
	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45\sqrt{E/F_y}$	
	6	Flexure in legs of single angles	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

TABLE 10.2.1 (cont.)
Limiting Width-Thickness Ratios for
Compression Elements

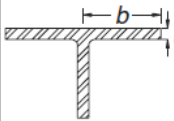
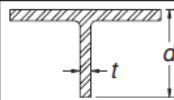
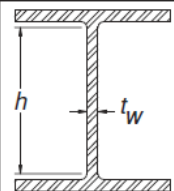
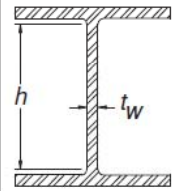
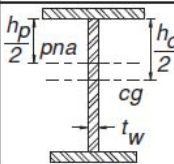
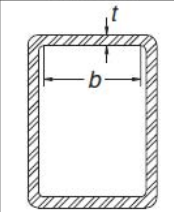
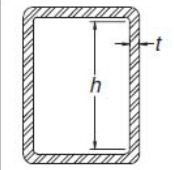
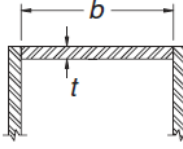
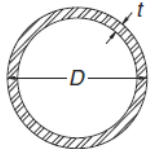
Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios		Example
			λ_p (compact)	λ_r (noncompact)	
7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
8	Uniform compression in stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$	
Stiffened Elements	9 Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
	10 Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49\sqrt{E/F_y}$	
	11 Flexure in webs of singly-symmetric I-shaped sections	h_c/t_w	$\frac{h_c}{h_p} \sqrt{\frac{E}{F_y}} \leq \lambda_r$ $\left(0.54 \frac{M_p}{M_y} - 0.09\right)^2 \leq \lambda_r$	$5.70\sqrt{E/F_y}$	
	12 Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
	13 Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	

TABLE 10.2.1 (cont.)
Limiting Width-Thickness Ratios for
Compression Elements

Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios		Example
			λ_p (compact)	λ_r (noncompact)	
14	Uniform compression in all other stiffened elements	b/t	NA	$1.49\sqrt{E/F_y}$	
15	Circular hollow sections In uniform compression	D/t	NA	$0.11 E/F_y$	
	In flexure	D/t	$0.07 E/F_y$	$0.31 E/F_y$	

[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)

[b] $F_L = 0.7F_y$ for minor-axis bending, major axis bending of slender-web built-up I-shaped members, and major axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_y S_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2)

In structures designed by elastic analysis, individual member stability and stability of the structure as a whole are provided jointly by:

- a) Calculation of the *required strengths* for members, connections and other elements using one of the methods specified in Section 10.3.2.2, and
- f) Satisfaction of the member and connection design requirements in this specification based upon those required strengths.

In structures designed by inelastic analysis, the provisions of Section 10.15 shall be satisfied.

10.3.1.1 Member Stability Design Requirements: Individual member stability is provided by satisfying the provisions of Section 10.5, 10.6, 10.7, 10.8, 10.10 and 10.11 etc. Where elements are designed to function as braces to define the unbraced length of columns and beams, the bracing system shall have sufficient stiffness and strength to control member movement at the braced points. Methods of satisfying this requirement are provided in Section 10.19.

10.3.1.2 System Stability Design Requirements

Lateral stability shall be provided by *moment frames, braced frames, shear walls*, and/or other equivalent *lateral load resisting systems*. The overturning effects of *drift* and the destabilizing influence of *gravity loads* shall be considered. *Force transfer and load sharing* between elements of the framing systems shall be considered. Braced-frame and shear-wall systems, moment frames, gravity framing systems, and *combined systems* shall satisfy the following specific requirements:

10.3.1.3.1 Braced-Frame and Shear-Wall Systems: In structures where lateral stability is provided solely by diagonal bracing, shear walls, or equivalent means, the *effective length factor, K*, for compression members shall be taken as 1.0, unless *structural analysis* indicates that a smaller value is appropriate. In braced-frame systems, it is permitted to design the columns, beams, and diagonal members as a vertically cantilevered, simply connected truss.

10.3.1.3.2 Moment Frame Systems: In frames where lateral stability is provided by the flexural stiffness of connected beams and columns, the effective length factor *K* or elastic critical buckling stress, F_e , for *columns and beam-columns* shall be determined as specified in Section 10.3.2.

10.3.1.3.3 Gravity Framing Systems: Columns in gravity framing systems shall be designed based on their actual length ($K = 1.0$) unless analysis shows that a smaller value may be used. The lateral stability of gravity framing systems shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. $P-\Delta$ effects due to load on the gravity columns shall be transferred to the lateral load resisting systems and shall be considered in the calculation of the required strengths of the lateral load resisting systems.

10.3.1.3.4 Combined Systems: The analysis and design of members, *connections* and other elements in combined systems of moment frames, braced frames, and/or shear walls and gravity frames shall meet the requirements of their respective systems.

10.3.2 Calculation of Required Strengths

Except as permitted in Section 10.3.2.2.2, *required strengths* shall be determined using a *second-order analysis* as specified in Section 10.3.2.1. Design by either second- order or *first-order analysis* shall meet the requirements specified in Section 10.3.2.2.

10.3.2.1 **Methods of Second-order Analysis:** Second-order analysis shall conform to the requirements in this Section.

10.3.2.1.1 General Second-Order Elastic Analysis

Any second-order elastic analysis method that considers both $P-\Delta$ and $P-\delta$ effects may be used. The Amplified First-Order Elastic Analysis Method defined in Section 10.3.2.1.2 is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

10.3.2.1.2 Second-Order Analysis by Amplified first-Order Elastic Analysis

The following is an approximate second-order analysis procedure for calculating the required flexural and axial strengths in members of *lateral load resisting systems*. The required second-order flexural strength, M_r , and axial strength, P_r , shall be determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \quad (10.3.2.1a)$$

$$P_r = P_{nt} + B_2 P_{lt} \quad (10.3.2.1b)$$

Where,

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \quad (10.3.2.2)$$

For members subjected to axial compression, B_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$.

For members in which $B_1 \leq 1.05$, it is conservative to amplify the sum of the non-sway and sway moments (as obtained, for instance, by a first-order elastic analysis) by the B_2 amplifier, in other words, $M_r = B_2 (M_{nt} + M_{lt})$.

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_e^2}} \geq 1 \quad (10.3.2.3)$$

And

$$\alpha = 1.00(\text{LRFD}) \quad \alpha = 1.60(\text{ASD})$$

M_r = required second-order flexural strength using LRFD or ASD load combinations, N-mm

M_{nt} = first-order moment using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N-mm

M_{lt} = first-order moment using LRFD or ASD load combinations caused by lateral translation of the frame only, N-mm

P_r = required second-order axial strength using LRFD or ASD load combinations, N

P_{nt} = first-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N

$\sum P_{nt}$ = total vertical load supported by the story using LRFD or ASD load combinations, including gravity column loads, N

P_{lt} = first-order axial force using LRFD or ASD load combinations caused by lateral translation of the frame only, N

C_m = a coefficient assuming no lateral translation of the frame whose value shall be taken as follows:

For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \quad (10.3.2.4)$$

where M_1 and M_2 , calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1 / M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

For beam-columns subjected to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway, N

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \quad (10.3.2.5)$$

ΣP_{e2} = elastic critical buckling resistance for the story determined by sideway buckling analysis, N

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is permitted to calculate the elastic story sidesway buckling resistance as

$$\Sigma P_{e2} = \Sigma \frac{\pi^2 EI}{(K_2 L)^2}$$

(10.3.2.6a)

For all types of lateral load resisting systems, it is permitted to use

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H}$$

(10.3.2.6b)

where

E = modulus of elasticity of steel = 200 000 MPa

R_M = 1.0 for braced-frame systems;

= 0.85 for moment-frame and combined systems, unless a larger value is justified by analysis

I = moment of inertia in the plane of bending, mm⁴

L = story height, mm

K_1 = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation, set equal to 1.0 unless analysis indicates that a smaller value may be used

K_2 = effective length factor in the plane of bending, calculated based on a sideway buckling analysis

Δ_H = first-order interstory drift due to lateral forces, mm. Where Δ_H varies over the plan area of the structure, Δ_H shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

ΣH = story shear produced by the lateral forces used to compute Δ_H , N

10.3.2.2 Design Requirements

These requirements apply to all types of braced, moment, and combined framing systems. Where the ratio of second-order drift to first-order drift is equal to or less than 1.5, the *required strengths* of members, *connections* and other elements shall be determined by one of the methods specified in Sections 10.3.2.2.1 or 10.3.2.2.2, or by the *Direct Analysis Method* of Section 10.14. Where the ratio of second-order drift to first-order drift is greater than 1.5, the required strengths shall be determined by the *Direct Analysis Method* of Section 10.14.

For the methods specified in Sections 10.3.2.2.1 or 10.3.2.2.2:

Analyses shall be conducted according to the design and loading requirements specified in either Section 10.2.3.3 (LRFD) or Section 10.2.3.4 (ASD).

The structure shall be analyzed using the nominal geometry and the nominal elastic stiffness for all elements.

10.3.2.2.1 Design By Second-Order Analysis

Where required strengths are determined by a *second-order analysis*:

The provisions of Section 10.3.2.1 shall be satisfied.

For design by ASD, analyses shall be carried out under 1.6 times the *ASD load combinations* and the results shall be divided by 1.6 to obtain the required strengths.

All gravity-only load combinations shall include a minimum lateral load applied at each level of the structure of $0.002Y_i$, where Y_i is the *design gravity load* applied at level i . This minimum *lateral load* shall be considered independently in two orthogonal directions.

Where the ratio of second-order drift to first-order drift is less than or equal to 1.1, members are permitted to be designed using $K = 1.0$. Otherwise, *columns* and *beam-columns* in *moment frames* shall be designed using a K factor or column buckling stress, F_e , determined from a sidesway buckling analysis of the structure. Stiffness reduction adjustment due to column inelasticity is permitted in the determination of the K factor. For *braced frames*, K for compression members shall be taken as 1.0, unless structural analysis indicates a smaller value may be used.

10.3.2.2.2 Design Requirements

Required strengths are permitted to be determined by a first-order analysis, with all members designed using $K = 1.0$, provided that

The required compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limitation:

$$\alpha P_r \leq 0.5P_y \quad (10.3.2.7)$$

Where,

$$\alpha = 1.0(\text{LRFD}) \quad \alpha = 1.6(\text{ASD})$$

P_r = required axial compressive strength under LRFD or ASD load combinations

P_y = member yield strength ($= AF_y$), N.

All load combinations include an additional lateral load, N_i , applied in combination with other loads at each level of the structure, where

$$N_i = 2.1(\Delta/L)Y_i \geq 0.0042Y_i \quad (10.3.2.8)$$

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i , N

Δ/L = the maximum ratio of Δ to L for all stories in the structure

Δ = first-order interstory drift due to the design loads, mm. Where Δ varies over the plan area of the structure, Δ shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

L = story height, mm

This additional lateral load shall be considered independently in two orthogonal directions.

The non-sway amplification of beam-column moments is considered by applying the B_1 amplifier of Section 10.3.2.1 to the total member moments.

10.4 Design of Members for Tension

This Section applies to steel members subject to axial tension caused by static *forces* acting through the centroidal axis.

10.4.1 Slenderness Limitations

The maximum slenderness (KL/r) limit for design of structural members (except cables and hanger rods) in tension shall be 300 unless it is justified by a comprehensive dynamic analysis (including 2nd order effects if applicable) that a higher slenderness ratio is satisfactory.

Here,

L = laterally unbraced length of the member, mm

r = governing radius of gyration, mm

K = the *effective length factor* determined in accordance with Section 10.3.2.

10.4.2 Tensile Strength

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t , of tension members, shall be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

- a) For tensile yielding in the gross section:

$$P_n = F_y A_g \quad (10.4.2.1)$$

$$\phi_t = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)}$$

- g) For tensile rupture in the net section:

$$P_n = F_u A_e \quad (10.4.2.2)$$

$$\phi_t = 0.75 \text{ (LRFD)} \quad \Omega_t = 2.00 \text{ (ASD)}$$

where

A_e = effective net area, mm²

A_g = gross area of member, mm²

F_y = specified minimum yield stress of the type of steel being used, MPa

F_u = specified minimum tensile strength of the type of steel being used, MPa

When members without holes are fully connected by welds, the effective net area used in Equation 10.4.2.2 shall be as defined in Section 10.4.3. When holes are present in a member with welded end connections, or at the welded connection in the case of plug or slot welds, the effective net area through the holes shall be used in Equation 10.4.2.2.

10.4.3 Area Determination

10.4.3.1 **Gross Area:** The gross area, A_g , of a member is the total cross-sectional area.

10.4.3.2 **Net Area:** The *net area*, A_n , of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as 2 mm greater than the *nominal dimension* of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section 10.10.3.2, of all holes in the chain, and adding, for each *gage* space in the chain, the quantity $s^2/(4g)$

where

s = longitudinal center-to-center spacing (*pitch*) of any two consecutive holes, mm.

g = transverse center-to-center spacing (*gage*) between *fastener* gage lines, mm.

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted *HSS* welded to a *gusset plate*, the net area, A_n , is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or *slot welds*, the *weld metal* shall not be considered as adding to the net area.

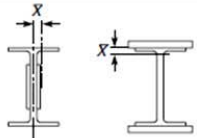
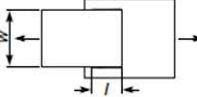
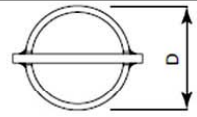
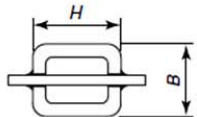
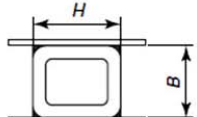
10.4.3.3 **Effective Net Area:** The effective area of tension members shall be determined as follows:

$$A_e = A_n U \quad (10.4.3.1)$$

where U , the shear lag factor, is determined as shown in Table 10.4.3.1.

Members such as single angles, double angles and WT sections shall have *connections* proportioned such that U is equal to or greater than 0.60. Alternatively, a lesser value of U is permitted if these tension members are designed for the effect of eccentricity in accordance with Sections 10.8.1.2 or 10.8.2.

TABLE 10.4.3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)		$U = 1.0$	
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \dots U = 1.0$ $2w > l \geq 1.5w \dots U = 0.87$ $1.5w > l \geq w \dots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \dots U = 1.0$ $D \leq l < 1.3D \dots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \dots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2 + 2BH}{4(B + H)}$	
		with two side gusset plates	$l \geq H \dots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2}{4(B + H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \dots U = 0.90$ $b_f < 2/3d \dots U = 0.85$	
		with web connected with 4 or more fasteners per line in the direction of loading	$U = 0.70$	
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	

l = length of connection, mm ; w = plate width, mm ; $\bar{x}$ = connection eccentricity, mm ; B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, mm ; H = overall height of rectangular HSS member, measured in the plane of the connection, mm

10.4.4 Built-Up Members

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.10.3.5.

Either perforated *cover plates* or *tie plates* without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or *fasteners* connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 150 mm.

10.4.5 Pin-Connected Members

10.4.5.1 Tensile Strength: The design tensile strength, $\phi_t P_n$ and the allowable tensile strength, P_n/Ω_t , of pin-connected members, shall be the lower value obtained according to the limit states of tensile rupture, shear rupture, bearing, and yielding.

- a) For tensile rupture on the net effective area:

$$P_n = 2t b_{eff} F_u \quad (10.4.5.1)$$

$$\phi_t = 0.75 \text{ (LRFD)} \quad \Omega_t = 2.00 \text{ (ASD)}$$

- h) For shear rupture on the effective area:

$$P_n = 0.6 F_u A_{sf} \quad (10.4.5.2)$$

$$\phi_{sf} = 0.75 \text{ (LRFD)} \quad \Omega_{sf} = 2.00 \text{ (ASD)}$$

where

$$A_{sf} = 2t (a + d/2), \text{ mm}^2$$

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, mm

$b_{eff} = 2t + 16, \text{ mm}$ but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

d = pin diameter, mm

t = thickness of plate, mm

- i) For bearing on the projected area of the pin, see Section 10.10.7.
j) For yielding on the gross section, use Equation 10.4.2.1.

10.4.5.2 Dimensional Requirements: The pin hole shall be located midway between the edges of the member in the direction normal to the applied *force*. When the pin is expected to provide for relative movement between connected parts while under full *load*, the diameter of the pin hole shall not be more than 1 mm greater than the diameter of the pin.

The width of the plate at the pin hole shall not be less than $2b_{eff} + d$ and the minimum extension, a , beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33 \times b_{eff}$.

The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the *net area* beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

10.4.6 Eyebars

10.4.6.1 Tensile Strength: The *available tensile strength* of *eyebars* shall be determined in accordance with Section 10.4.2, with A_g taken as the cross-sectional area of the body.

For calculation purposes, the width of the body of the *eyebars* shall not exceed eight times its thickness.

10.4.6.2 Dimensional Requirements: *Eyebars* shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads with the periphery concentric with the pin hole.

The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.

The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin hole diameter shall not be more than 1 mm greater than the pin diameter.

For steels having F_y greater than 485 MPa, the hole diameter shall not exceed five times the plate thickness and the width of the eyebar body shall be reduced accordingly.

A thickness of less than 13 mm is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width from the hole edge to the plate edge perpendicular to the direction of applied *load* shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

10.5 Design of Members for Compression

This Section addresses members subject to axial compression through the centroidal axis.

10.5.1 General Provisions

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n / Ω_c , are determined as follows:

The nominal compressive strength, P_n , shall be the lowest value obtained according to the limit states of flexural buckling, torsional buckling and flexural-torsional buckling.

For doubly symmetric and singly symmetric members the limit state of flexural buckling is applicable.

For singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, the limit states of torsional or flexural-torsional buckling are also applicable.

$\phi_c = 0.90$ (LRFD) $\Omega_c = 1.67$ (ASD)

10.5.2 Slenderness Limitations and effective Length

The effective length factor, K , for calculation of column slenderness, KL/r , shall be determined in accordance with Section 10.3,

where

L = laterally unbraced length of the member, mm

r = governing radius of gyration, mm

K = the *effective length factor* determined in accordance with Section 10.3.2.

The maximum limit of slenderness, KL/r , for compression members shall be 150 unless a comprehensive analysis including second order effects (including dynamic effects if any) shows that a higher value is justified.

10.5.3 Compressive Strength for Flexural Buckling of Members without Slender elements

This section applies to compression members with compact and noncompact sections, as defined in Section 10.2.4, for uniformly compressed elements.

The nominal compressive strength, P_n , shall be determined based on the limit state of flexural buckling.

$$P_n = F_{cr} A_g \quad (10.5.3.1)$$

The flexural buckling stress, F_{cr} , is determined as follows:

$$\begin{aligned} \text{(a) When } \frac{KL}{r} &\leq 4.71 \sqrt{\frac{E}{F_y}} \quad (\text{or } F_e \geq 0.44 F_y) \\ &= [0.658] \quad (10.5.3.2) \end{aligned}$$

$$\begin{aligned} \text{(b) When } \frac{KL}{r} &> 4.71 \sqrt{\frac{E}{F_y}} \quad (\text{or } F_e < 0.44 F_y) \\ &= 0.877 \quad (10.5.3.3) \end{aligned}$$

where

F_e = elastic critical buckling stress determined according to Equation 10.5.3.4, Section 10.5.4, or the provisions of Section 10.3.2, as applicable,

$$= \frac{2}{\left(\frac{0}{+}\right)^2} \quad (10.5.3.4)$$

10.5.4 Compressive Strength for Torsional and Flexural-Torsional Buckling of Members without Slender elements

This section applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up *columns* with *compact* and *noncompact sections*, as defined in Section 10.2.4 for uniformly compressed elements. These provisions are not required for single angles, which are covered in Section 10.5.5.

The *nominal compressive strength*, P_n , shall be determined based on the *limit states* of *flexural-torsional* and *torsional buckling*, as follows:

$$P_n = F_{cr} A_g \quad (10.5.4.1)$$

For double-angle and tee-shaped compression members:

$$= \left(\frac{+}{2}\right) \left[1 - \sqrt{1 - \frac{4}{(+)^2}} \right] \quad (10.5.4.2)$$

where F_{cry} is taken as F_{cr} from Equation 10.5.3.2 or 10.5.3.3, for *flexural buckling*

about the y-axis of symmetry and $\frac{KL}{r} = \frac{KL}{r_y}$, and

$$= \frac{2}{0} \quad (10.5.4.3)$$

For all other cases, F_{cr} shall be determined according to Equation 10.5.3.2 or 10.5.3.3, using the torsional or flexural-torsional elastic buckling stress, F_e , determined as follows:

For doubly symmetric members:

$$= \left[\frac{2}{\left(\frac{0}{+}\right)^2} + \right] \frac{1}{+} \quad (10.5.4.4)$$

For singly symmetric members where y is the axis of symmetry:

$$= \left(\frac{+}{2}\right) \left[1 - \sqrt{1 - \frac{4}{(+)^2}} \right] \quad (10.5.4.5)$$

For unsymmetric members, F_e is the lowest root of the cubic equation:

$$\left(\frac{-}{-}\right) \left(\frac{-}{-}\right) \left(\frac{-}{-}\right) - 2 \left(\frac{-}{-}\right) \left(\frac{0}{-}\right)^2 - 2 \left(\frac{-}{-}\right) \left(\frac{0}{-}\right)^2 = 0 \quad (10.5.4.6)$$

where

A_g = gross area of member, mm²

C_w = warping constant, mm⁶

$$\frac{2}{0} = \frac{2}{0} + \frac{2}{0} + \frac{+}{+} \quad (10.5.4.7)$$

$$= 1 - \frac{\frac{2}{0} + \frac{2}{0}}{\frac{2}{0}} \quad (10.5.4.8)$$

$$= \frac{2}{\left(\frac{0}{+}\right)^2} \quad (10.5.4.9)$$

$$= \frac{2}{\left(\frac{0}{+}\right)^2} \quad (10.5.4.10)$$

$$= \left(\frac{2}{\left(\frac{0}{+}\right)^2} + \right) \frac{1}{\frac{2}{0}} \quad (10.5.4.11)$$

G = shear modulus of elasticity of steel = 77 200 MPa

I_x, I_y = moment of inertia about the principal axes, mm⁴

J = torsional constant, mm⁴

K_z = *effective length factor* for torsional buckling

x_o, y_o = coordinates of shear center with respect to the centroid, mm

$\bar{r}_0$ = polar radius of gyration about the shear center, mm

r_y = radius of gyration about y-axis, mm

10.5.5 Single Angle Compression Members

The nominal compressive strength, P_n , of single angle members shall be determined in accordance with Section 10.5.3 or Section 10.5.7, as appropriate, for axially loaded members, as well as those subject to the slenderness modification of Section 10.5.5(a) or 10.5.5(b), provided the members meet the criteria imposed.

The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that: (1) members are loaded at the ends in compression through the same one leg; (2) members are attached by welding or by minimum two-bolt connections; and (3) there are no intermediate transverse loads.

- a) For equal-leg angles or unequal-leg angles connected through the longer leg that are individual members or are web members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord:

$$\begin{aligned} \text{(i) When } 0 \leq \frac{L}{r_x} \leq 80 \\ = 72 + 0.75 \cdot \frac{L}{r_x} \end{aligned} \quad (10.5.5.1)$$

$$\begin{aligned} \text{(ii) When } \frac{L}{r_x} > 80 \\ = 32 + 1.25 \cdot \frac{L}{r_x} \leq 200 \end{aligned} \quad (10.5.5.2)$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Equations 10.5.5.1 and 10.5.5.2 shall be increased by adding $4[(b_l/b_s)^2 - 1]$, but KL/r of the members shall not be less than $0.95L/r_z$.

- k) For equal-leg angles or unequal-leg angles connected through the longer leg that are web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord:

$$\begin{aligned} \text{(i) When } 0 \leq \frac{L}{r_x} \leq 75 \\ = 60 + 0.8 \cdot \frac{L}{r_x} \end{aligned} \quad (10.5.5.3)$$

$$\begin{aligned} \text{(ii) When } \frac{L}{r_x} > 75 \\ = 45 + 1.25 \cdot \frac{L}{r_x} \leq 200 \end{aligned} \quad (10.5.5.4)$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Equations 10.5.5.3 and 10.5.5.4 shall be increased by adding $6[(b_l/b_s)^2 - 1]$, but KL/r of the members shall not be less than $0.82L/r_z$.

where

L = length of member between work points at truss chord centerlines, mm

b_l = longer leg of angle, mm

b_s = shorter leg of angle, mm

r_x = radius of gyration about geometric axis parallel to connected leg, mm

r_z = radius of gyration for the minor principal axis, mm

Single angle members with different end conditions from those described in Section 10.5.5(a) or (b), with leg length ratios greater than 1.7, or with transverse loading shall be evaluated for combined axial load and flexure using the provisions of Section 10.8. End connection to different legs on each end or to both legs, the use of single bolts or the attachment of adjacent web members to opposite sides of the gusset plate or chord shall constitute different end conditions requiring the use of Section 10.8 provisions.

10.5.6 Built-up Members

10.5.6.1 Compressive Strength

- a) The nominal compressive strength of built-up members composed of two or more shapes that are interconnected by bolts or welds shall be determined in accordance with Sections 10.5.3, 10.5.4, or 10.5.7 subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, KL/r is replaced by $(KL/r)_m$ determined as follows:

- (i) For intermediate connectors that are snug-tight bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_0^2 + \left(\frac{a}{r_i}\right)^2} \quad (10.5.6.1)$$

- (ii) For intermediate connectors that are welded or pretensioned bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_0^2 + 0.82 \frac{a^2}{(1+\alpha^2) r_i^2}} \quad (10.5.6.2)$$

Where

$\left(\frac{KL}{r}\right)_m$ = modified column slenderness of built-up member

$\left(\frac{KL}{r}\right)_0$ = column slenderness of built-up member acting as a unit in the buckling direction being considered

a = distance between connectors, mm

r_i = minimum radius of gyration of individual component, mm

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm

α = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the member axis of buckling, mm

- l) The nominal compressive strength of built-up members composed of two or more shapes or plates with at least one open side interconnected by perforated cover plates or lacing with tie plates shall be determined in accordance with Sections 10.5.3, 10.5.4, or 10.5.7 subject to the modification given in Section 10.5.6.1 (a).

10.5.6.2 Dimensional Requirements

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a , such that the effective slenderness ratio Ka/r_i of each of the component shapes, between the fasteners, does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration, r_i , shall be used in computing the slenderness ratio of each component part. The end connection shall be welded or pre-tensioned bolted with Class A or B faying surfaces.

At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to $1\frac{1}{2}$ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds or bolts shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.10.3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 305 mm, when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 460 mm.

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section 10.2.4, is assumed to contribute to the available strength provided the following requirements are met:

The width-thickness ratio shall conform to the limitations of Section 10.2.4.

The ratio of length (in direction of stress) to width of hole shall not exceed two.

The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.

The periphery of the holes at all points shall have a minimum radius of 38 mm.

As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing available strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that the L/r ratio of the flange included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2 percent of the available compressive strength of the member. The L/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, L is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70 percent of that distance for double lacing.

For additional spacing requirements, see Section 10.10.3.5.

10.5.7 Members with Slender Elements

This section applies to compression members with slender sections, as defined in Section 10.2.4 for uniformly compressed elements.

The *nominal compressive strength*, P_n , shall be determined based on the *limit states of flexural, torsional and flexural-torsional buckling*.

$$P_n = F_{cr} A_g \quad (10.5.7.1)$$

$$\begin{aligned} \text{a) When } \frac{KL}{r} &\leq 4.71 \sqrt{\frac{E}{QF_y}} \quad (\text{or } F_e \geq 0.44 QF_y) \\ &= [0.658] \end{aligned} \quad (10.5.7.2)$$

$$\begin{aligned} \text{m) When } \frac{KL}{r} &> 4.71 \sqrt{\frac{E}{QF_y}} \quad (\text{or } F_e < 0.44 QF_y) \\ &= 0.877 \end{aligned} \quad (10.5.7.3)$$

Where

F_e = elastic critical buckling stress, calculated using Equations 10.5.3.4 and 10.5.4.4 for doubly symmetric members, Equations 10.5.3.4 and 10.5.4.5 for singly symmetric members, and Equation 10.5.4.6 for unsymmetric members, except for single angles where F_e is calculated using Equation 10.5.3.4.

$Q = 1.0$ for members with *compact* and *noncompact* sections, as defined in Section 10.2.4, for uniformly compressed elements

$= Q_s Q_a$ for members with *slender-element sections*, as defined in Section 10.2.4, for uniformly compressed elements.

10.5.7.1 Slender Unstiffened Elements, Q_s

The reduction factor Q_s for slender *unstiffened elements* is defined as follows:

For flanges, angles, and plates projecting from rolled *columns* or other compression members:

$$\begin{aligned} \text{When } \frac{b}{t} &\leq 0.56 \sqrt{\frac{E}{F_y}} \\ &= 1.0 \end{aligned} \quad (10.5.7.4)$$

When $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$

$$= 1.415 - 0.74 \left(\right) \sqrt{\quad} \quad (10.5.7.5)$$

When $b/t \geq 1.03\sqrt{E/F_y}$

$$= \frac{0.69}{\left(\right)^2} \quad (10.5.7.6)$$

For flanges, angles, and plates projecting from built-up columns or other compression members:

When $\frac{b}{t} \leq 0.64 \sqrt{\frac{Ek_c}{F_y}}$

$$= 1.0 \quad (10.5.7.7)$$

When $0.64 \sqrt{\frac{Ek_c}{F_y}} < b/t \leq 1.17 \sqrt{\frac{Ek_c}{F_y}}$

$$= 1.415 - 0.65 \left(\right) \sqrt{\quad} \quad (10.5.7.8)$$

when $\frac{b}{t} > 1.17 \sqrt{\frac{Ek_c}{F_y}}$

$$= \frac{0.90}{\left(\right)^2} \quad (10.5.7.9)$$

Where $k_c = \frac{4}{\sqrt{h/t_w}}$, and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

For single angles

When $\frac{b}{t} \leq 0.45 \sqrt{\frac{E}{F_y}}$

$$= 1.0 \quad (10.5.7.10)$$

When $0.45\sqrt{E/F_y} < b/t \leq 0.91\sqrt{E/F_y}$

$$= 1.34 - 0.76 \left(\right) \sqrt{\quad} \quad (10.5.7.11)$$

When $b/t > 0.91\sqrt{E/F_y}$

$$= \frac{0.53}{\left(\right)^2} \quad (10.5.7.12)$$

Where

b = full width of longest angle leg, mm

For stems of tees

When $\frac{d}{t} \leq 0.75 \sqrt{\frac{E}{F_y}}$

$$= 1.0 \quad (10.5.7.13)$$

When $0.75 \sqrt{\frac{E}{F_y}} < d/t \leq 1.03 \sqrt{\frac{E}{F_y}}$

$$= 1.908 - 1.22 \left(\right) \sqrt{\quad} \quad (10.5.7.14)$$

When $d/t > 1.03 \sqrt{\frac{E}{F_y}}$

$$= \frac{0.69}{\left(\right)^2} \quad (10.5.7.15)$$

Where

b = width of unstiffened compression element, as defined in Section 10.2.4, mm

d = the full nominal depth of tee, mm

t = thickness of element, mm

10.5.7.2 Slender Unstiffened Elements, Q_s

The reduction factor, Q_a , for slender stiffened elements is defined as follows:

$$Q_a = \frac{A}{A_{eff}} \quad (10.5.7.16)$$

A = total cross-sectional area of member, mm²

A_{eff} = summation of the effective areas of the cross section based on the reduced effective width, b_e , mm²

The reduced effective width, b_e , is determined as follows:

For uniformly compressed slender elements, with $\frac{b}{t} \geq 1.49 \sqrt{\frac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

$$b_e = 1.92 \sqrt{\left[1 - \frac{0.34}{(\lambda)}\right]} \leq b \quad (10.5.7.17)$$

Where

λ is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$.

For flanges of square and rectangular slender-element sections of uniform thickness with $\frac{b}{t} \geq 1.40 \sqrt{\frac{E}{f}}$

$$b_e = 1.92 \sqrt{\left[1 - \frac{0.38}{(\lambda)}\right]} \leq b \quad (10.5.7.18)$$

Where $f = P_n / A_{eff}$

For axially-loaded circular sections:

$$\begin{aligned} \text{When } 0.11 \frac{E}{F_y} < \frac{D}{t} < 0.45 \frac{E}{F_y} \\ b_e = \frac{0.038}{(\lambda)} + \frac{2}{3} \end{aligned} \quad (10.5.7.19)$$

Where

D = outside diameter, mm

t = wall thickness, mm

10.6 Design of Members for Flexure

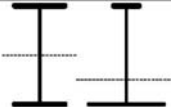
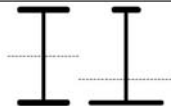
This section applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at *load* points and supports.

10.6.1 General Provisions

The design flexural strength, $\phi_b M_n$, and the allowable flexural strength, M_n / Ω_b , shall be determined as follows:

- a) For all provisions in this Section 10.6
 - $\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)
 - and the nominal flexural strength, M_n , shall be determined according to Sections 10.6.2 through 10.6.12.
- n) The provisions in this chapter are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis.

TABLE 10.6.1.1
Selection Table for the Application
of Sub-sections of Section 10.6

Sub-section in this provision	Cross Section	Flange Slenderness	Web Slenderness	Limit States
10.6.2		C	C	Y, LTB
10.6.3		NC, S	C	LTB, FLB
10.6.4		C, NC, S	C, NC	Y, LTB, FLB, TFY
10.6.5		C, NC, S	S	Y, LTB, FLB, TFY
10.6.6		C, NC, S	N/A	Y, FLB
10.6.7		C, NC, S	C, NC	Y, FLB, WLB
10.6.8		N/A	N/A	Y, LB
10.6.9		C, NC, S	N/A	Y, LTB, FLB
10.6.10		N/A	N/A	Y, LTB, LLB
10.6.11		N/A	N/A	Y, LTB
10.6.12	Unsymmetrical shapes	N/A	N/A	All limit states

Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender

The following terms are common to the equations in this chapter except where noted:

C_b = lateral-torsional buckling modification factor for non-uniform moment diagrams when both ends of the unsupported segment are braced

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} R_m \leq 3.0 \quad (10.6.1.1)$$

Where

$M_{\max}$ = absolute value of maximum moment in the unbraced segment, N-mm

M_A = absolute value of moment at quarter point of the unbraced segment, N-mm

M_B = absolute value of moment at centerline of the unbraced segment, N-mm

M_C = absolute value of moment at three-quarter point of the unbraced segment, N-mm

R_m = cross-section monosymmetry parameter

= 1.0, doubly symmetric members

= 1.0, singly symmetric members subjected to single curvature bending

= $0.5 + 2 \left(\frac{I_{yc}}{I_y} \right)^2$, singly symmetric members subjected to reverse curvature bending

I_y = moment of inertia about the principal y-axis, mm⁴

I_{yc} = moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending, referred to the smaller flange, mm⁴

In singly symmetric members subjected to *reverse curvature* bending, the *lateral-torsional buckling* strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

C_b is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$.

10.6.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section 10.2.4.

The *nominal flexural strength*, M_n , shall be the lower value obtained according to the *limit states of yielding (plastic moment)* and *lateral-torsional buckling*.

10.6.2.1 Yielding

$$M_n = M_p = F_y Z_x \quad (10.6.2.1)$$

where

F_y = specified minimum yield stress of the type of steel being used, MPa

Z_x = plastic section modulus about the x-axis, mm³

10.6.2.2 Lateral –Torsional Buckling

a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.

o) When $L_p < L_b \leq L_r$

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (10.6.2.2)$$

p) When $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \quad (10.6.2.3)$$

Where

L_b = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_0} \left(\frac{L_b}{r_{ts}} \right)^2} \quad (10.6.2.4)$$

and where

E = modulus of elasticity of steel = 200 000 MPa

J = torsional constant, mm⁴

S_x = elastic section modulus taken about the x-axis, mm³

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76r_y \sqrt{\frac{E}{F_y}} \quad (10.6.2.5)$$

$$L_r = 1.95r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y S_x h_o}{E Jc} \right)^2}} \quad (10.6.2.6)$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad (10.6.2.7)$$

and

$$\text{For a doubly symmetric I-shape: } c = 1 \quad (10.6.2.8a)$$

$$\text{For a channel: } c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}} \quad (10.6.2.8b)$$

Where

h_o = distance between the flange centroids, mm

10.6.3 Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis

This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section 10.2.4.

The nominal flexural strength, M_n , shall be the lower value obtained according to the *limit states* of *lateral-torsional buckling* and compression flange *local buckling*.

10.6.3.1 Lateral –Torsional Buckling

For *lateral-torsional buckling*, the provisions of Section 10.6.2.2 shall apply.

10.6.3.2 Compression Flange Local Buckling

a) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (10.6.3.1)$$

q) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \quad (10.6.3.2)$$

Where

$$\lambda = \frac{b_f}{2t_f}$$

$\lambda_{pf} = \lambda_p$ is the limiting slenderness for a compact flange, Table 10.2.4.1

$\lambda_{rf} = \lambda_r$ is the limiting slenderness for a noncompact flange, Table 10.2.4.1

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

10.6.4 Other I-Shaped Members with Compact or Noncompact Webs Bent about Their Major Axis

This section applies to: (a) doubly symmetric I-shaped members bent about their major axis with noncompact webs; and (b) singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section B4.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the *limit states* of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

10.6.4.1 Compression Flange Yielding

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc} \quad (10.6.4.1)$$

10.6.4.2 Lateral-Torsional Buckling

- a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.
r) When $L_p < L_b \leq L_r$

$$M_n = C_b \left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc}) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq R_{pc} M_{yc} \quad (10.6.4.2)$$

- s) When $L_b > L_r$

$$M_n = F_{cr} S_{xc} \leq R_{pc} M_{yc} \quad (10.6.4.3)$$

Where

$$M_{yc} = F_y S_{xc} \quad (10.6.4.4)$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_0} \left(\frac{L_b}{r_t} \right)^2} \quad (10.6.4.5)$$

For $\frac{I_{yc}}{I_y} \leq 0.23$, J shall be taken as zero.

The stress, F_L , is determined as follows:

For $\frac{S_{xt}}{S_{xc}} \geq 0.7$

$$F_L = 0.7 F_y \quad (10.6.4.6a)$$

For $\frac{S_{xt}}{S_{xc}} < 0.7$

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5 F_y \quad (10.6.4.6b)$$

The limiting laterally unbraced length for the limit state of yielding, L_p is,

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \quad (10.6.4.7)$$

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, L_r , is

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_0}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L S_{xc} h_0}{E J} \right)^2}} \quad (10.6.4.8)$$

The web plastification factor, R_{pc} , is determined as follows:

For $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \quad (10.6.4.9a)$$

For $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1 \right) \left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yc}} \quad (10.6.4.9b)$$

Where

$$M_p = Z_x F_y \leq 1.6 S_{xc} F_y$$

S_{xc}, S_{xt} = elastic section modulus referred to tension and compression flanges, respectively, mm³

$$\lambda = \frac{h_c}{t_w}$$

$\lambda_{pw} = \lambda_p$ limiting slenderness for a compact web, Table 10.2.4.1

$\lambda_{rw} = \lambda_r$ limiting slenderness for a noncompact web, Table 10.2.4.1

The effective radius of gyration for lateral-torsional buckling, r_t , is determined as follows:

For I-shapes with a rectangular compression flange:

$$r_t = \frac{b_{fc}}{\sqrt{12 \left(\frac{h_0}{d} + \frac{1}{6} a_w \frac{h^2}{h_0 d} \right)}} \quad (10.6.4.10)$$

Where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \quad (10.6.4.11)$$

b_{fc} = compression flange width, mm

t_{fc} = compression flange thickness, mm

For I-shapes with channel caps or cover plates attached to the compression flange:

r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, mm

a_w = the ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components.

10.6.4.1 Compression Flange Local Buckling

- a) For sections with compact flanges, the *limit state of local buckling* does not apply.
- t) For sections with noncompact flanges

$$M_n = \left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc}) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (10.6.4.12)$$

- u) For sections with slender flanges

$$M_n = \frac{0.9 E k_c S_{xc}}{\lambda^2} \quad (10.6.4.13)$$

Where

F_L is defined in Equations 10.6.4.6a and 10.6.4.6b

R_{pc} = is the web plastification factor, determined by Equations 10.6.4.9

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$$\lambda = \frac{b_{fc}}{2 t_{fc}}$$

$\lambda_{pf} = \lambda_p$ limiting slenderness for a compact flange, Table 10.2.4.1

$\lambda_{rf} = \lambda_r$ limiting slenderness for a noncompact flange, Table 10.2.4.1

10.6.4.4 Tension Flange Yielding

- a) When $S_{xt} \geq S_{xc}$, the *limit state of tension flange yielding* does not apply.
- v) When $S_{xt} < S_{xc}$

$$M_n = R_{pt} M_{yt} \quad (10.6.4.14)$$

Where

$$M_{yt} = F_y S_{xt}$$

The web *plastification* factor corresponding to the tension flange yielding limit state, R_{pt} , is determined as follows:

For $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}} \quad (10.6.4.15a)$$

For $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1 \right) \left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yt}} \quad (10.6.4.15b)$$

Where

$$\lambda = \frac{h_c}{t_w}$$

$\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, defined in Table 10.2.4.1

$\lambda_{rw} = \lambda_r$, the limiting slenderness for a noncompact web, defined in Table 10.2.4.1

10.6.5 Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges, bent about their major axis, as defined in Section 10.2.4.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the *limit states* of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

10.6.5.1 Compression Flange Yielding

$$M_n = R_{pg} F_y S_{xc} \quad (10.6.5.1)$$

10.6.5.2 Lateral-Torsional Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad (10.6.5.2)$$

- a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.
- w) When $L_p < L_b \leq L_r$

$$F_{cr} = C_b \left[F_y - (0.3 F_y) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq F_y \quad (10.6.5.3)$$

- x) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \leq F_y \quad (10.6.5.4)$$

where

L_p is defined by Equation 10.6.4.7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7 F_y}} \quad (10.6.5.5)$$

R_{pg} is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300 a_w} \left(\frac{h_c}{t_w} - 5.7 \sqrt{\frac{E}{F_y}} \right) \leq 1.0 \quad (10.6.5.6)$$

a_w is defined by Equation 10.6.4.11 but shall not exceed 10 and

r_t is the effective radius of gyration for lateral buckling as defined in Section 10.6.4.

10.6.5.3 Compression Flange Local Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad (10.6.5.7)$$

- a) For sections with compact flanges, the limit state of compression flange local buckling does not apply.
 y) For sections with noncompact flanges

$$F_{cr} = \left[F_y - (0.3 F_y) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (10.6.5.8)$$

- z) For sections with slender flanges

$$F_{cr} = \frac{0.9 E k_c}{\left(\frac{b_f}{2 t_f} \right)^2} \quad (10.6.5.9)$$

Where

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$$\lambda = \frac{b_{fc}}{2 t_{fc}}$$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table 10.2.4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table 10.2.4.1

10.6.5.4 Tension Flange Yielding

(a) When $S_{xt} \geq S_{xc}$, the *limit state* of tension flange *yielding* does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \quad (10.6.5.10)$$

10.6.6 I-Shaped Members and Channels Bent about Their Minor Axis

This section applies to I-shaped members and channels bent about their minor axis.

The nominal flexural strength, M_n , shall be the lower value obtained according to the *limit states of yielding* (*plastic moment*) and *flange local buckling*.

10.6.6.1 Yielding

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad (10.6.6.1)$$

10.6.6.2 Flange Local Buckling

- a) For sections with compact flanges the limit state of yielding shall apply.
 aa) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7 F_y S_y) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (10.6.6.2)$$

- bb) For sections with slender flanges

$$M_n = F_{cr} S_y \quad (10.6.6.3)$$

where

$$F_{cr} = \frac{0.69 E}{\left(\frac{b_f}{2 t_f} \right)^2} \quad (10.6.6.4)$$

$$\lambda = \frac{b}{t}$$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table 10.2.4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table 10.2.4.1

S_y for a channel shall be taken as the minimum section modulus

10.6.7 Square and Rectangular HSS and Box-Shaped Members

This section applies to square and rectangular *HSS*, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section 10.2.4.

The nominal flexural strength, M_n , shall be the lowest value obtained according to the *limit states* of *yielding* (*plastic moment*), *flange local buckling* and *web local buckling* under pure flexure.

10.6.7.1 Yielding

$$M_n = M_p = F_y Z \quad (10.6.7.1)$$

Where

Z = plastic section modulus about the axis of bending, mm^3

10.6.7.2 Flange Local Buckling

- a) For *compact sections*, the *limit state* of *flange local buckling* does not apply. (b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_y S) \left(3.57 \frac{b}{t} \sqrt{\frac{F_y}{E}} - 4.0 \right) \leq M_p \quad (10.6.7.2)$$

- cc) For sections with slender flanges

$$M_n = F_y S_{eff}$$

(10.6.7.3)

where

S_{eff} is the *effective section modulus* determined with the *effective width* of the compression flange taken as:

$$b_e = 1.92t \sqrt{\frac{E}{F_y}} \left[1 - \frac{0.38}{b/t} \sqrt{\frac{E}{F_y}} \right] \leq b \quad (10.6.7.4)$$

10.6.7.3 Web Local Buckling

- a) For compact sections, the limit state of web local buckling does not apply.
dd) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S_x) \left(0.305 \frac{h}{t_w} \sqrt{\frac{F_y}{E}} - 0.738 \right) \leq M_p \quad (10.6.7.5)$$

10.6.8 Round HSS

This section applies to round *HSS* having D/t ratios of less than $\frac{0.45E}{F_y}$

The nominal flexural strength, M_n , shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and *local buckling*.

10.6.8.1 Yielding

$$M_n = M_p = F_y Z \quad (10.6.8.1)$$

10.6.8.2 Local Buckling

- a) For *compact sections*, the *limit state* of *flange local buckling* does not apply. (b) For noncompact sections

$$M_n = \left(\frac{0.021E}{\frac{D}{t}} + F_y \right) S \quad (10.6.8.2)$$

ee) For sections with slender walls

$$M_n = F_{cr} S \quad (10.6.8.3)$$

where

$$F_{cr} = \frac{0.33E}{\frac{D}{t}} \quad (10.6.8.4)$$

S = elastic section modulus, mm³

10.6.9 Tees and Double Angles Loaded in the Plane of Symmetry

This section applies to tees and double angles loaded in the plane of symmetry. The nominal flexural strength, M_n , shall be the lowest value obtained according to the *limit states of yielding (plastic moment), lateral-torsional buckling* and *flange local buckling*.

10.6.9.1 Yielding

$$M_n = M_p \quad (10.6.9.1)$$

Where

$$M_p = F_y Z_x \leq 1.6 M_y \text{ for stems in tension} \quad (10.6.9.2)$$

$$\leq M_y \quad \text{for stems in compression} \quad (10.6.9.3)$$

10.6.9.2 Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi \sqrt{EI_y GJ}}{L_b} \left[B + \sqrt{1 + B^2} \right] \quad (10.6.9.4)$$

Where

$$B = \pm 2.3 \left(\frac{d}{L_b} \right) \sqrt{\frac{I_y}{J}} \quad (10.6.9.5)$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, the negative value of B shall be used.

10.6.9.3 Flange Local Buckling of Tees

$$M_n = F_{cr} S_{xc} \quad (10.6.9.6)$$

S_{xc} is the elastic section modulus referred to the compression flange.

F_{cr} is determined as follows:

For compact sections, the limit state of flange local buckling does not apply.

For noncompact sections

$$F_{cr} = F_y \left(1.19 - 0.50 \left(\frac{b_f}{2t_f} \right) \sqrt{\frac{F_y}{E}} \right) \quad (10.6.9.7)$$

For slender sections

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f} \right)^2} \quad (10.6.9.8)$$

10.6.10 Single Angle

This section applies to single angles with and without continuous lateral restraint along their length.

Single angles with continuous lateral-torsional restraint along the length shall be permitted to be designed on the basis of *geometric axis* (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for *principal axis* bending except where the provision for bending about a geometric axis is permitted.

The *nominal flexural strength*, M_n , shall be the lowest value obtained according to the *limit states* of *yielding* (*plastic moment*), *lateral-torsional buckling* and *leg local buckling*.

10.6.10.1 Yielding

$$M_n = 1.5 M_y \quad (10.6.10.1)$$

where

M_y = *yield moment* about the axis of bending, N-mm

10.6.10.2 Lateral-Torsional Buckling

For single angles without continuous lateral-torsional restraint along the length

When $M_e \leq M_y$

$$M_n = \left(0.92 - \frac{0.17 M_e}{M_y} \right) M_e \quad (10.6.10.2)$$

When $M_e > M_y$

$$M_n = \left(1.92 - 1.17 \sqrt{\frac{M_y}{M_e}} \right) M_y \leq 1.5 M_y \quad (10.6.10.3)$$

Where

M_e , the elastic *lateral-torsional buckling* moment, is determined as follows:

For bending about one of the *geometric axes* of an equal-leg angle with no lateral-torsional restraint

With maximum compression at the toe

$$M_e = \frac{0.66 E b^4 t C_b}{L^2} \left(\sqrt{1 + 0.78 \left(\frac{L t}{b^2} \right)^2} - 1 \right) \quad (10.6.10.4a)$$

With maximum tension at the toe

$$M_e = \frac{0.66 E b^4 t C_b}{L^2} \left(\sqrt{1 + 0.78 \left(\frac{L t}{b^2} \right)^2} + 1 \right) \quad (10.6.10.4b)$$

M_y shall be taken as 0.80 times the *yield moment* calculated using the geometric section modulus.

For bending about one of the geometric axes of an equal-leg angle with lateral-torsional restraint at the point of maximum moment only

M_e shall be taken as 1.25 times M_e computed using Equation 10.6.10.4a or 10.6.10.4b.

M_y shall be taken as the yield moment calculated using the geometric section modulus.

For bending about the major principal axis of equal-leg angles:

$$M_e = \frac{0.46 E b^2 t^2 C_b}{L} \quad (10.6.10.5)$$

For bending about the major principal axis of unequal-leg angles:

$$M_e = \frac{4.9 E I_z C_b}{L^2} \left(\sqrt{\beta_w^2 + 0.052 \left(\frac{L t}{r_z} \right)^2} + \beta_w \right) \quad (10.6.10.6)$$

Where

C_b is computed using Equation 10.6.1.1 with a maximum value of 1.5.

L = laterally *unbraced length* of a member, mm

I_z = minor principal axis moment of inertia, mm⁴

r_z = radius of gyration for the minor principal axis, mm

t = angle leg thickness, mm

β_w = a section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

10.6.10.3 Leg Local Buckling

The *limit state of leg local buckling* applies when the toe of the leg is in compression.

- a) For *compact sections*, the limit state of leg local buckling does not apply.
- ff) For sections with noncompact legs

$$M_n = F_y S_c \left(2.43 - 1.72 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \right) \quad (10.6.10.7)$$

For sections with slender legs

$$M_n = F_{cr} S_c \quad (10.6.10.8)$$

Where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t} \right)^2} \quad (10.6.10.9)$$

b = outside width of leg in compression, mm

S_c = elastic section modulus to the toe in compression relative to the axis of bending, mm³. For bending about one of the *geometric axes* of an equal-leg angle with no lateral-torsional restraint, S_c shall be 0.80 of the geometric axis section modulus.

10.6.11 Rectangular Bars and Rounds

This section applies to rectangular bars bent about either *geometric axis* and rounds.

The *nominal flexural strength*, M_n , shall be the lower value obtained according to the *limit states of yielding* (*plastic moment*) and *lateral-torsional buckling*, as required.

10.6.11.1 Yielding

For rectangular bars with $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$ bent about their major axis, rectangular bars bent about their minor axis and rounds:

$$M_n = M_p = F_y Z \leq 1.6 M_y \quad (10.6.11.1)$$

10.6.11.2 Lateral-Torsional Buckling

- a) For rectangular bars with $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \frac{F_y}{E} \right] M_y \leq M_p \quad (10.6.11.2)$$

- gg) For rectangular bars with $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = F_{cr} S_x \leq M_p \quad (10.6.11.3)$$

Where

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \quad (10.6.11.4)$$

t = width of rectangular bar parallel to axis of bending, mm

d = depth of rectangular bar, mm

L_b = length between points that are either braced against lateral displacement of the compression region or braced against twist of the cross section, mm

(c) For rounds and rectangular bars bent about their minor axis, the *limit state* of *lateral-torsional buckling* need not be considered.

10.6.12 Unsymmetrical Shapes

This section applies to all unsymmetrical shapes, except single angles.

The *nominal flexural strength*, M_n , shall be the lowest value obtained according to the *limit states* of *yielding* (yield moment), *lateral-torsional buckling* and *local buckling* where

$$M_n = F_n S \quad (10.6.12.1)$$

Where

S = lowest elastic section modulus relative to the axis of bending, mm³

10.6.12.1 Yielding

$$F_n = F_y \quad (10.6.12.2)$$

10.6.12.2 Lateral-Torsional Buckling

$$F_n = F_{cr} \leq F_y \quad (10.6.12.3)$$

Where

F_{cr} = buckling stress for the section as determined by analysis, MPa

10.6.12.3 Local Buckling

$$F_n = F_{cr} \leq F_y \quad (10.6.12.4)$$

Where

F_{cr} = buckling stress for the section as determined by analysis, MPa

10.6.13 Proportions of Beams and Girders

10.6.13.1 Hole reductions

This section applies to rolled or built-up shapes, and cover-plated *beams* with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the *limit states* specified in other sections of this Chapter, the *nominal flexural strength*, M_n , shall be limited according to the limit state of *tensile rupture* of the tension flange.

For $F_u A_{fn} \geq Y_t F_y A_{fg}$, the limit state of tensile rupture does not apply.

For $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n , at the location of the holes in the tension flange shall not be taken greater than:

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \quad (10.6.13.1)$$

Where

A_{fg} = gross tension flange area, calculated in accordance with the provisions of Section 10.4.3.1, mm²

A_{fn} = net tension flange area, calculated in accordance with the provisions of Section 10.4.3.2, mm²

Y_t = 1.0 for $F_y / F_u \leq 0.8$

= 1.1 otherwise

10.6.13.2 Proportioning Limits for I-Shaped Members

Singly symmetric I-shaped members shall satisfy the following limit:

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \quad (10.6.13.2)$$

I-shaped members with slender webs shall also satisfy the following limits:

For $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 11.7 \sqrt{\frac{E}{F_y}} \quad (10.6.13.3)$$

For $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.42E}{F_y} \quad (10.3.13.4)$$

Where

a = clear distance between transverse stiffeners, mm

In unstiffened girders h/t_w shall not exceed 260. The ratio of the web area to the compression flange area shall not exceed 10.

10.6.13.3 Cover Plates

Flanges of welded *beams* or girders may be varied in thickness or width by splicing a series of plates or by the use of *cover plates*.

The total cross-sectional area of cover plates of bolted girders shall not exceed 70 percent of the total flange area.

High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total *horizontal shear* resulting from the bending *forces* on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear.

However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section 10.5.6 or 10.4.4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any *loads* applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical *connection* or *fillet welds*. The attachment shall be adequate, at the applicable strength given in Sections 10.10.2.2, 10.10.3.8, or 10.2.3.9 to develop the cover plate's portion of the flexural strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length a' , defined below, and shall be adequate to develop the cover plate's portion of the strength of the beam or girder at the distance a' from the end of the cover plate.

When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \quad (10.6.13.5)$$

Where

w = width of cover plate, mm.

When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \quad (10.6.13.6)$$

When there is no weld across the end of the plate

$$a' = 2w \quad (10.6.13.7)$$

10.6.13.4 Built-Up Beams

Where two or more *beams* or channels are used side-by-side to form a flexural member, they shall be connected together in compliance with Section 10.5.6.2. When concentrated *loads* are carried from one beam to another, or distributed between the beams, *diaphragms* having sufficient *stiffness* to distribute the load shall be welded or bolted between the beams.

10.7 Design of Members for Shear

This section addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and *HSS* sections, and shear in the weak direction of singly or doubly symmetric shapes.

10.7.1 General Provisions

Two methods of calculating shear strength are presented below. The method presented in Section 10.7.2 does not utilize the post *buckling strength* of the member (*tension field action*). The method presented in Section 10.7.3 utilizes tension field action.

The *design shear strength*, $\phi_v V_n$ and the *allowable shear strength*, V_n/Ω_v , shall be determined as follows.

For all provisions in this section except Section 10.7.2.1a:

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

10.7.2 Members with Unstiffened or Stiffened Webs

10.7.2.1 Nominal Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n , of unstiffened or stiffened webs, according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6F_y A_w C_w \quad (10.7.2.1)$$

For webs of rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$

$$\phi_v = 1.00 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$

And

$$C_v = 1.0 \quad (10.7.2.2)$$

For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v , is determined as follows:

For $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \quad (10.7.2.3)$$

For $1.10\sqrt{k_v E/F_y} < h/t_w \leq 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad (10.7.2.4)$$

For $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \quad (10.7.2.5)$$

Where

A_w = the overall depth times the web thickness, dt_w , mm^2

The web plate buckling coefficient, k_v , is determined as follows:

For unstiffened webs with $h/t_w < 260$, $k_v = 5$ except for the stem of tee shapes where $k_v = 1.2$.

For stiffened webs,

$$k_v = 5 + \frac{5}{(a/h)^2}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)} \right]^2$$

Where

a = clear distance between transverse *stiffeners*, mm

h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, mm

= for built-up welded sections, the clear distance between flanges, mm

= for built-up bolted sections, the distance between *fastener* lines, mm

= for tees, the overall depth, mm.

10.7.2.2 Transverse Stiffeners

Transverse *stiffeners* are not required where $h/t_w \leq 2.46\sqrt{E/F_y}$, or where the required shear strength is less than or equal to the available shear strength provided in accordance with Section 10.7.2.1 for $k_v = 5$.

Transverse *stiffeners* used to develop the available web shear strength, as provided in Section 10.7.2.1, shall have a moment of inertia about an axis in the web center for *stiffener* pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \quad (10.7.2.6)$$

Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When *lateral bracing* is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit 1 percent of the total flange *force*, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 305 mm on center. If intermittent *fillet welds* are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 250 mm.

10.7.3 Tension Field Action

10.7.3.1 Limits on the Use of Tension Field Action

Consideration of *tension field action* is permitted for flanged members when the web plate is supported on all four sides by flanges or *stiffeners*. Consideration of tension field action is not permitted for:

end panels in all members with transverse stiffeners;

members when a/h exceeds 3.0 or $[260/(h/t_w)]^2$;

$$2 A_w / (A_{fc} + A_{ft}) > 2.5; \text{ or}$$

$$h/b_{fc} \text{ or } h/b_{ft} > 6.0$$

where

A_{fc} = area of compression flange, mm²

A_{ft} = area of tension flange, mm²

b_{fc} = width of compression flange, mm

b_{ft} = width of tension flange, mm

In these cases, the nominal shear strength, V_n , shall be determined according to the provisions of Section 10.7.2.

10.7.3.2 Nominal Shear Strength with Tension Field Action

When *tension field action* is permitted according to Section 10.7.3.1, the nominal shear strength, V_n , with tension field action, according to the *limit state* of tension field *yielding*, shall be

$$\text{For } h/t_w \leq 1.10\sqrt{k_v E/F_y}$$

$$V_n = 0.6F_y A_w \quad (10.7.3.1)$$

For $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \quad (10.7.3.2)$$

Where

k_v and C_v are as defined in Section 10.7.2.1.

10.7.3.3 Transverse Stiffeners

Transverse *stiffeners* subject to *tension field action* shall meet the requirements of Section 10.7.2.2 and the following limitations:

$$(b/t)_{st} \leq 0.56 \sqrt{\frac{E}{F_{yst}}} \quad (10.7.3.3)$$

$$A_{st} > \frac{F_y}{F_{yst}} \left[0.15D_s h t_w (1 - C_v) \frac{V_r}{V_c} - 18t_w^2 \right] \geq 0$$

Where

$(b/t)_{st}$ = the width-thickness ratio of the stiffener

F_{yst} = specified minimum yield stress of the stiffener material, MPa

C_v = coefficient defined in Section 10.7.2.1

D_s = 1.0 for stiffeners in pairs

= 1.8 for single angle stiffeners

= 2.4 for single plate stiffeners

V_r = required shear strength at the location of the stiffener, N

V_c = available shear strength; $\phi_v V_n$ (LRFD) or V_n/Ω_v (ASD) with V_n as defined in Section 10.7.3.2, N

10.7.4 Single Angles

The nominal shear strength, V_n , of a single angle leg shall be determined using Equation 10.7.2.1 with $C_v = 1.0$, $A_w = bt$ where b = width of the leg resisting the shear force, mm and $k_v = 1.2$.

10.7.5 Rectangular HSS and Box Members

The nominal shear strength, V_n , of rectangular HSS and box members shall be determined using the provisions of Section 10.7.2.1 with $A_w = 2ht$ where h for the width resisting the shear force shall be taken as the clear distance between the flanges less the inside corner radius on each side and $t_w = t$ and $k_v = 5$. If the corner radius is not known, h shall be taken as the corresponding outside dimension minus three times the thickness.

10.7.6 Round HSS

The nominal shear strength, V_n , of round HSS, according to the limit states of shear yielding and shear buckling, is

$$V_n = F_{cr} A_g / 2 \quad (10.7.6.1)$$

Where

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D} \left(\frac{D}{t} \right)^{\frac{5}{4}}}} \quad (10.7.6.2a)$$

and

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \quad (10.7.6.2b)$$

but shall not exceed $0.6 F_y$

A_g = gross area of section based on design wall thickness, mm^2

D = outside diameter, mm

L_v = the distance from maximum to zero shear force, mm

T = design wall thickness, equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal thickness for SAW HSS, mm

10.7.7 Weak Axis Shear in Singly and Doubly Symmetric Shapes

For singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength, V_n , for each shear resisting element shall be determined using Equation 10.7.2.1 and Section 10.7.2.1b with $A_w = b_f t_f$ and $k_v = 1.2$.

10.7.8 Beams and girders with Web Openings

The effect of all web openings on the nominal shear strength of steel and *composite beams* shall be determined. Adequate reinforcement shall be provided when the *required strength* exceeds the *available strength* of the member at the opening.

10.8 Design of Members for Combined Forces and Torsion

This section addresses members subject to axial *force* and flexure about one or both axes, with or without torsion, and to members subject to torsion only.

10.8.1 Doubly and Singly Symmetric Members Subject to Flexure and Axial Force

10.8.1.1 Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which $0.1 \leq (I_{yc} / I_y) \leq 0.9$, that are constrained to bend about a geometric axis (x and/or y) shall be limited by Equations 10.8.1.1a and 10.8.1.1b, where I_{yc} is the moment of inertia about the y-axis referred to the compression flange, mm^4 .

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (10.8.1.1a)$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (10.8.1.1b)$$

Where

P_r = required axial compressive strength, N

P_c = available axial compressive strength, N

M_r = required flexural strength, N-mm

M_c = available flexural strength, N-mm

x = subscript relating symbol to strong axis bending

y = subscript relating symbol to weak axis bending

For design according to Section 10.2.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

$P_c = \phi_c P_n$ = design axial compressive strength, determined in accordance with Section 10.5, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Section 10.6, N-mm

ϕ_c = resistance factor for compression = 0.90

ϕ_b = resistance factor for flexure = 0.90

For design according to Section 10.2.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

$P_c = P_n / \Omega_c$ = allowable axial compressive strength, determined in accordance with Section 10.5, N

M_r = required flexural strength using ASD load combinations, N-mm

$M_c = M_n / \Omega_b$ = allowable flexural strength determined in accordance with Section 10.6, N-mm

Ω_c = safety factor for compression = 1.67

Ω_b = safety factor for flexure = 1.67

10.8.1.2 Doubly and Singly Symmetric Members in Flexure and Tension

The interaction of flexure and tension in doubly symmetric members and singly symmetric members constrained to bend about a geometric axis (x and/or y) shall be limited by Equations 10.8.1.1a and 10.8.1.1b,

Where

For design according to Section 10.2.3.3 (LRFD)

P_r = required tensile strength using LRFD load combinations, N

$P_c = \phi_t P_n$ = design tensile strength, determined in accordance with Section 10.4.2, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Section 10.6, N-mm

ϕ_t = resistance factor for tension (see Section 10.4.2)

ϕ_b = resistance factor for flexure = 0.90

For doubly symmetric members, C_b in Section 10.6 may be increased by $\sqrt{1 + \frac{P_u}{P_{ey}}}$ for axial tension that acts concurrently with flexure,

Where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$$

For design according to Section 10.2.3.4 (ASD)

P_r = required tensile strength using ASD load combinations, N

$P_c = P_n / \Omega_t$ = allowable tensile strength, determined in accordance with Section 10.4.2, N

M_r = required flexural strength using ASD load combinations, N-mm

$M_c = M_n / \Omega_b$ = allowable flexural strength determined in accordance with Section 10.6, N-mm

Ω_t = safety factor for tension (see Section 10.4.2)

Ω_b = safety factor for flexure = 1.67

For doubly symmetric members, C_b in Section 10.6 may be increased by $\sqrt{1 + \frac{1.5P_u}{P_{ey}}}$ for axial tension that acts concurrently with flexure

Where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$$

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations 10.8.1.1a and 10.8.1.1b.

10.8.1.3 Doubly Symmetric Members in Single Axis Flexure and Compression

For doubly symmetric members in flexure and compression with moments primarily in one plane, it is permissible to consider the two independent limit states, in-plane instability and out-of-plane buckling or flexural-torsional buckling, separately in lieu of the combined approach provided in Section 10.8.1.1.

- a) For the limit state of in-plane instability, Equations 10.8.1.1 shall be used with P_c , M_r , and M_c determined in the plane of bending.
- hh) For the limit state of out-of-plane buckling

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}} \right)^2 \leq 1.0 \quad (10.8.1.2)$$

Where

P_{co} = available compressive strength out of the plane of bending, N

M_{cx} = available flexural-torsional strength for strong axis flexure determined from Section 10.6, N-mm

If bending occurs only about the weak axis, the moment ratio in Equation 10.8.1.2 shall be neglected.

For members with significant biaxial moments ($M_r/M_c \geq 0.05$ in both directions), the provisions of Section 10.8.1.1 shall be followed.

10.8.2 Unsymmetric and Other Members Subject to Flexure and Axial Force

This section addresses the interaction of flexure and axial *stress* for shapes not covered in Section 10.8.1. It is permitted to use the provisions of this Section for any shape in lieu of the provisions of Section 10.8.1.

$$\left| \frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \right| \leq 1.0 \quad (10.8.2.1)$$

Where

f_a = required axial stress at the point of consideration, MPa

F_a = available axial stress at the point of consideration, MPa

f_{bw}, f_{bz} = required flexural stress at the point of consideration, MPa

F_{bw}, F_{bz} = available flexural stress at the point of consideration, MPa

w = subscript relating symbol to major principal axis bending

z = subscript relating symbol to minor principal axis bending

For design according to Section 10.2.3.3 (LRFD)

f_a = required axial stress using LRFD load combinations, MPa

$F_a = \phi_c F_{cr}$ = design axial stress, determined in accordance with Section 10.5 for compression or Section 10.4.2 for tension, MPa

f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section using LRFD load combinations, MPa

$F_{bw}, F_{bz} = \frac{\phi_b M_n}{S} = \text{design flexural stress determined in accordance with Section 10.6, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress.}$

ϕ_c = resistance factor for compression = 0.90

ϕ_t = resistance factor for tension (Section 10.4.2)

ϕ_b = resistance factor for flexure = 0.90

For design according to Section 10.2.3.4 (ASD)

f_a = required axial stress using ASD load combinations, MPa

$F_a = \frac{F_{cr}}{\Omega_c} = \text{allowable axial stress determined in accordance with Section 10.5 for compression, or Section 10.4.2 for tension, MPa}$

f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section using ASD load combinations, MPa

$F_{bw}, F_{bz} = \frac{M_n}{\Omega_b S} = \text{allowable flexural stress determined in accordance with Section 10.6, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress.}$

Ω_c = safety factor for compression = 1.67

Ω_t = safety factor for tension (Section 10.4.2)

Ω_b = safety factor for flexure = 1.67

Equation 10.8.2.1 shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as appropriate. When the axial *force* is compression, second order effects shall be included according to the provisions of Section 10.3.

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equation 10.8.2.1.

10.8.3 Members under Torsion and Combined Torsion, Flexure, Shear and/or Axial Force

10.8.3.1 Torsional Strength of Round and Rectangular HSS

The design torsional strength, $\phi_T T_n$ and the allowable torsional strength, T_n/Ω_T , for round and rectangular HSS shall be determined as follows:

$\phi_T = 0.90$ (LRFD) $\Omega_T = 1.67$ (ASD)

The nominal torsional strength, T_n , according to the limit states of torsional yielding and torsional buckling is:

$$T_n = F_{cr} C \quad (10.8.3.1)$$

Where

C is the HSS torsional constant

F_{cr} shall be determined as follows:

For round HSS, F_{cr} shall be the larger of

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D} \left(\frac{D}{t}\right)^{\frac{5}{4}}}} \quad (10.8.3.2a)$$

and

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \quad (10.8.3.2b)$$

but shall not exceed $0.6 F_y$,

where

L = length of the member, mm

D = outside diameter, mm

For rectangular HSS

For $h/t \leq 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \quad (10.8.3.3)$$

For $2.45\sqrt{E/F_y} < h/t \leq 3.07\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \left(2.45\sqrt{E/F_y}\right)/(h/t) \quad (10.8.3.4)$$

For $3.07\sqrt{E/F_y} < h/t \leq 260$

$$F_{cr} = 0.458\pi^2 E/(h/t)^2 \quad (10.8.3.5)$$

10.8.3.2 HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

When the *required torsional strength*, T_r , is less than or equal to 20 percent of the *available torsional strength*, T_c , the interaction of torsion, shear, flexure and/or axial *force* for HSS shall be determined by Section 10.8.1 and the torsional effects shall be neglected. When T_r exceeds 20 percent of T_c , the interaction of torsion, shear, flexure and/or axial *force* shall be limited by

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0 \quad (10.8.3.6)$$

Where

For design according to Section 10.2.3.3 (LRFD)

P_r = required axial strength using LRFD load combinations, N

$P_c = \phi P_n$, design tensile or compressive strength in accordance with Section 10.4 or 10.5, N

M_r = required flexural strength using LRFD load combinations, N-mm

$M_c = \phi_b M_n$, design flexural strength in accordance with Section 10.6, N-mm

V_r = required shear strength using LRFD load combinations, N

$V_c = \phi_v V_n$, design shear strength in accordance with Section 10.7, N

T_r = required torsional strength using LRFD load combinations, N-mm

$T_c = \phi_T T_n$, design torsional strength in accordance with Section 10.8.3.1, N-mm

For design according to Section 10.2.3.4 (ASD)

P_r = required axial strength using ASD load combinations, N

$P_c = P_n / \Omega$, allowable tensile or compressive strength in accordance with Section 10.4 or 10.5, N

M_r = required flexural strength using ASD load combinations determined in accordance with Section 10.2.5, N-mm

$M_c = M_n / \Omega_b$, allowable flexural strength in accordance with Section 10.6, N-mm

V_r = required shear strength using ASD load combinations, N

$V_c = V_n / \Omega_v$, allowable shear strength in accordance with Section 10.7, N

T_r = required torsional strength using ASD load combinations, N-mm

$T_c = T_n / \Omega_T$, allowable torsional strength in accordance with Section 10.8.3.1, N-mm

10.8.3.3 Strength of Non-HSS Members under Torsion and Combined Stress

The design torsional strength, $\phi_T F_n$, and the allowable torsional strength, F_n / Ω_T , for non-HSS members shall be the lowest value obtained according to the limit states of yielding under normal stress, shear yielding under shear stress, or buckling, determined as follows:

$\phi_T = 0.90$ (LRFD) $\Omega_T = 1.67$ (ASD)

a) For the limit state of yielding under normal stress

$$F_n = F_y \quad (10.8.3.7)$$

ii) For the limit state of shear yielding under shear stress

$$F_n = 0.6 F_y \quad (10.8.3.8)$$

jj) For the limit state of buckling

$$F_n = F_{cr} \quad (10.8.3.9)$$

where

F_{cr} = buckling stress for the section as determined by analysis, MPa

Some constrained local yielding is permitted adjacent to areas that remain elastic.

10.9 EVALUATION OF EXISTING STRUCTURES

This section applies to the evaluation of the strength and stiffness under static vertical (gravity) loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section 10.1.3.1. This section does not address load testing for the effects of seismic loads or moving loads (vibrations).

10.9.1 GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the available strength of a load resisting

member or system. The evaluation shall be performed by *structural analysis* (Section 10.9.3), by *load tests* (Section 10.9.4), or by a combination of *structural analysis* and load tests, as specified in the contract documents. Where load tests are used, the *engineer of record* shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

10.9.2 MATERIAL PROPERTIES

Determination of Required Tests: The *engineer of record* shall determine the specific tests that are required from Section 10.9.2.2 through 10.9.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

10.9.2.1 Tensile Properties

Tensile properties of members shall be considered in evaluation by *structural analysis* (Section 10.9.3) or *load tests* (Section 10.9.4). Such properties shall include the *yield stress*, *tensile strength* and *percent elongation*. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

10.9.2.2 Chemical Composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

10.9.2.3 Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Section 10.1.3.1d are critical to the performance of the structure, the *Charpy V-Notch toughness* shall be determined in accordance with the provisions of Section 10.1.3.1d. If the notch toughness so determined does not meet the provisions of Section 10.1.3.1d, the *engineer of record* shall determine if remedial actions are required.

10.9.2.4 Weld Metal

Where structural performance is dependent on existing welded *connections*, representative samples of *weld metal* shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of AWS D1.1 are not met, the *engineer of record* shall determine if remedial actions are required.

10.9.2.5 Bolts and Rivets

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine *tensile strength* in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 shall be permitted. Rivets shall be assumed to be ASTM A502, Grade 1, unless a higher grade is established through documentation or testing.

10.9.3 EVALUATION BY STRUCTURAL ANALYSIS

10.9.3.1 Dimensional Data

All dimensions used in the evaluation, such as spans, *column* heights, member spacings, bracing locations, cross section dimensions, thicknesses and *connection* details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

10.9.3.2 Strength Evaluation

Forces (load effects) in members and connections shall be determined by *structural analysis* applicable to the type of structure evaluated. The load effects shall be determined for the *loads* and *factored load combinations* stipulated in Section 10.2.2.

The *available strength* of members and connections shall be determined from applicable provisions of Sections 10.2 through 10.19 of this Specification.

10.9.3.3 Serviceability Evaluation

Where required, the deformations at *service loads* shall be calculated and reported.

10.9.4 EVALUATION BY LOAD TESTS

10.9.4.1 Determination of Load Rating by Testing

To determine the *load* rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the *engineer of record's* plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to $1.2D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures, L_r , S , or R as defined in the Symbols, shall be substituted for L . More severe *load combinations* shall be used if required by specifications of Chapter 2 of Part 6.

Periodic unloading shall be considered once the *service load* level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

10.9.4.2 Serviceability Evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

10.9.5 EVALUATION REPORT

After the evaluation of an existing structure has been completed, the *engineer of record* shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by *structural analysis*, by *load* testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, mill test reports and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the structure, including all members and *connections*, is adequate to withstand the *load effects*.

10.10 Connections

This section addresses connecting elements, connectors, and the affected elements of the connected members not subject to fatigue loads.

10.10.1 General Provisions

10.10.1.1 Design Basis

The *design strength*, ϕR_n , and the *allowable strength* R_n / Ω , of *connections* shall be determined in accordance with the provisions of this section and the provisions of Section 10.2.

The *required strength* of the connections shall be determined by *structural analysis* for the specified *design loads*, consistent with the type of construction specified, or shall be a proportion of the required strength of the connected members when so specified herein.

Where the gravity axes of intersecting axially loaded members do not intersect at one point, the effects of eccentricity shall be considered.

10.10.1.2 Simple Connection

Simple connections of *beams*, *girders*, or *trusses* shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic, but self-limiting deformation in the *connection* is permitted to accommodate the end rotation of a simple beam.

10.10.1.3 Moment Connection

End *connections* of restrained *beams*, *girders*, and *trusses* shall be designed for the combined effect of *forces* resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section 10.2.6.3.2.

10.10.1.4 Compression Members with Bearing Joints

- a) When *columns* bear on bearing plates or are finished to bear at *splices*, there shall be sufficient connectors to hold all parts securely in place.
- b) When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for either (i) or (ii) below. It is permissible to use the less severe of the two conditions:
 - (i) An axial tensile *force* of 50 percent of the required compressive strength of the member; or
 - (ii) The moment and shear resulting from a transverse *load* equal to 2 percent of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

10.10.1.5 Splices in Heavy Sections

When tensile *forces* due to applied tension or flexure are to be transmitted through *splices* in heavy sections, as defined in Section 10.1.3.1c and 10.1.3.1d, by complete- joint-penetration groove (CJP) welds, material notch-toughness requirements as given in Section 10.1.3.1c and 10.1.3.1d, weld access hole details as given in Section 10.10.1.6 and thermal cut surface preparation and inspection requirements as given in 10.1.3.2.2 shall apply. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

10.10.1.6 Beam Copes and Weld Access Holes

All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than $1\frac{1}{2}$ times the thickness of the material in which the hole is made. The height of the access hole shall be $1\frac{1}{2}$ times the thickness of the material with the access hole, t_w , but not less than 25 mm nor does it need to exceed 50 mm. The access hole shall be detailed to provide room for weld backing as needed.

For sections that are rolled or welded prior to cutting, the edge of the web shall be sloped or curved from the surface of the flange to the *reentrant* surface of the access hole. In hot-rolled shapes, and built-up shapes with CJP *groove welds* that join the web-to-flange, all *beam copes* and weld access holes shall be free of notches and sharp reentrant corners. No arc of the weld access hole shall have a radius less than 10 mm.

In built-up shapes with fillet or *partial-joint-penetration groove welds* that join the web-to-flange, all beam copes and weld access holes shall be free of notches and sharp reentrant corners. The access hole shall be

permitted to terminate perpendicular to the flange, providing the weld is terminated at least a distance equal to the weld size away from the access hole.

For heavy sections as defined in 10.1.3.1c and 10.1.3.1d, the *thermally cut* surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of *splice* welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

10.10.1.7 Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial *force* into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically loaded single angle, double angle, and similar members.

10.10.1.8 Bolts in Combination with Welds

Bolts shall not be considered as sharing the load in combination with welds, except that shear connections with any grade of bolts permitted by Section 10.1.3.3 installed in standard holes or short slots transverse to the direction of the load are permitted to be considered to share the load with longitudinally loaded fillet welds. In such connections the available strength of the bolts shall not be taken as greater than 50 percent of the available strength of bearing-type bolts in the connection.

In making welded alterations to structures, existing rivets and high strength bolts tightened to the requirements for *slip-critical connections* are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional required strength.

10.10.1.9 High-Strength Bolts in Combination with Rivets

In both new work and alterations, in *connections* designed as *slip-critical connections* in accordance with the provisions of Section 10.10.3, high-strength bolts are permitted to be considered as sharing the *load* with existing rivets.

10.10.1.10 Limitations on Bolted and Welded Connections

Pretensioned joints, *slip-critical joints* or welds shall be used for the following *connections*:

Column splices in all multi-story structures over 38 m in height

Connections of all *beams* and *girders* to columns and any other beams and girders on which the bracing of columns is dependent in structures over 38 m in height

In all structures carrying cranes of over 50 kN capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports

Connections for the support of machinery and other live *loads* that produce impact or reversal of load

Snug-tightened joints or joints with ASTM A307 bolts shall be permitted except where otherwise specified.

10.10.2 Welds

All provisions of AWS D1.1 apply under this Specification, with the exception that the provisions of the listed sections apply under this specification in lieu of the cited AWS provisions as follows:

Section 10.10.1.6 in lieu of AWS D1.1 Section 5.17.1

Section 10.10.2.2.1 in lieu of AWS D1.1 Section 2.3.2

Table 10.10.2.2 in lieu of AWS D1.1 Table 2.1

Table 10.10.2.5 in lieu of AWS D1.1 Table 2.3

Table 10.17.1.1 in lieu of AWS D1.1 Table 2.4

Section 10.2.3.9 and Section 10.17 in lieu of AWS D1.1 Section 2, Part C

Section 10.13.2.2 in lieu of AWS D1.1 Sections 5.15.4.3 and 5.15.4.4

10.10.2.1 Groove Welds

10.10.2.1.1 Effective Area

The effective area of *groove welds* shall be considered as the length of the weld times the effective throat thickness.

The effective throat thickness of a *complete-joint-penetration (CJP) groove weld* shall be the thickness of the thinner part joined.

Table 10.10.2.1: Effective Throat of Partial-Joint-Penetration Groove Welds

Welding Process	Welding Position F (flat), H (horiz.), V (vert.), OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded Metal Arc (SMAW)	All	J or U Groove	Depth of Groove
Gas Metal Arc (GMAW)	All	60° V	
Flux Cored Arc (FCAW)			
Submerged Arc (SAW)	F	J or U Groove 60° Bevel or V	Depth of Groove Minus 3 mm
Gas Metal Arc (GMAW)	F, H	45° Bevel	
Flux Cored Arc (FCAW)			
Shielded Metal Arc (SMAW)	All	45° Bevel	Depth of Groove Minus 3 mm
Gas Metal Arc (GMAW)	V, OH	45° Bevel	
Flux Cored Arc (FCAW)			

The effective throat thickness of a *partial-joint-penetration (PJP) groove weld* shall be as shown in Table 10.10.2.1.

The effective weld size for flare groove welds, when filled flush to the surface of a round bar, a 90° bend in a *formed section*, or rectangular *HSS* shall be as shown in Table 10.10.2.2, unless other effective throats are demonstrated by tests. The effective size of flare groove welds filled less than flush shall be as shown in Table 10.10.2.2, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

TABLE 10.10.2.2: Effective Weld Sizes of Flare Groove Welds

Welding Process	Flare Bevel Groove ^[a]	Flare V Groove
GMAW and FCAW-G	$\frac{5}{8}R$	$\frac{3}{4}R$
SMAW and FCAW-S	$\frac{5}{16}R$	$\frac{5}{8}R$
SAW	$\frac{5}{16}R$	$\frac{1}{2}R$

[a] For Flare Bevel Groove with $R < 10$ mm use only reinforcing fillet weld on filled flush joint. General Note: R = radius of joint surface (can be assumed to be $2t$ for HSS), mm

Larger effective throat thicknesses than those in Table 10.10.2.2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

10.10.2.1.2 Limitations

The minimum effective throat thickness of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated *forces* nor the size shown in Table 10.10.2.3. Minimum weld size is determined by the thinner of the two parts joined.

TABLE 10.10.2.3: Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material Thickness of Thinner Part Joined, mm	Minimum Effective Throat Thickness, ^[a] mm
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19 to 38	8
Over 38 to 57	10
Over 57 to 150	13

[a] See Table 10.10.2.1.

10.10.2.2 Fillet Welds

10.10.2.2.1 Effective Area

The effective area of a *fillet weld* shall be the *effective length* multiplied by the effective throat. The effective throat of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. An increase in effective throat is permitted if consistent penetration beyond the root of the diagrammatic weld is demonstrated by tests using the production process and procedure variables.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the *faying surface*.

10.10.2.2.2 Limitations

The minimum size of fillet welds shall be not less than the size required to transmit calculated *forces* nor the size as shown in Table 10.10.2.4. These provisions do not apply to *fillet weld reinforcements of partial- or complete-joint-penetration groove welds*.

TABLE 10.10.2.4: Minimum Size of Fillet Welds

Material Thickness of Thinner	Minimum Size of Fillet ^[a]
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6

[a] Leg dimension of fillet welds. Single pass welds must be used.

The maximum size of fillet welds of connected parts shall be:

Along edges of material less than 6 mm thick, not greater than the thickness of the material.

Along edges of material 6 mm or more in thickness, not greater than the thickness of the material minus 2 mm, unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 2 mm provided the weld size is clearly verifiable.

The minimum effective length of fillet welds designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed 1/4 of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section 10.4.3.3.

For end-loaded fillet welds with a length up to 100 times the leg dimension, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β ,

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \quad (10.10.2.1)$$

where

L = actual length of end-loaded weld, mm

w = weld leg size, mm

When the length of the weld exceeds 300 times the leg size, the value of β shall be taken as 0.60.

Intermittent fillet welds are permitted to be used to transfer calculated *stress* across a *joint* or *faying surfaces* when the *required strength* is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of *built-up members*. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of 38 mm.

In *lap joints*, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 25 mm. Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Fillet weld terminations are permitted to be stopped short or extend to the ends or sides of parts or be boxed except as limited by the following:

For lap joints in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.

For *connections* where flexibility of the outstanding elements is required, when end returns are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.

Fillet welds joining *transverse stiffeners* to *plate girder* webs 19 mm thick or less shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of *stiffeners* are welded to the flange.

Fillet welds that occur on opposite sides of a common plane shall be interrupted at the corner common to both welds.

Fillet welds in holes or slots are permitted to be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section 10.10.2. Fillet welds in holes or slots are not to be considered *plug* or *slot welds*.

10.10.2.3 Plug and Slot Welds

10.10.2.3.1 Effective Area

The effective shearing area of *plug* and *slot welds* shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the *faying surface*.

10.10.2.3.2 Limitations

Plug or slot welds are permitted to be used to transmit shear in *lap joints* or to prevent buckling of lapped parts and to join component parts of *built-up members*.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus 8 mm, rounded to the next larger odd even mm, nor greater than the minimum diameter plus 3 mm or $2\frac{1}{4}$ times the thickness of the weld.

The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 8 mm rounded to the next larger odd even mm, nor shall it be larger than $2\frac{1}{4}$ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material 16 mm or less in thickness shall be equal to the thickness of the material. In material over 16 mm thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than 16 mm.

10.10.2.4 Strength

The *design strength*, ϕR_n and the allowable strength, R_n / Ω , of welds shall be the lower value of the base material and the weld metal strength determined according to the *limit states of tensile rupture, shear rupture or yielding* as follows:

TABLE 10.10.2.5: Available Strength of Welded Joints, N

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_w)	Effective Area (A_{BM} or A_w) mm ²	Required Filler Metal Strength Level ^{[a][b]}
COMPLETE-JOINT-PENETRATION GROOVE WELDS					
Tension Normal to weld axis	Strength of the joint is controlled by the base metal				Matching filler metal shall be used. For T and corner joints with backing left in place, notch tough filler metal is required. See Section 10.10.2.6.
Compression Normal to weld axis	Strength of the joint is controlled by the base metal				Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Strength of the joint is controlled by the base metal				Matching filler metal shall be used. ^[c]
PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS					
Tension Normal to weld axis	$\phi = 0.90$		See		Filler metal with a strength level equal to or less than matching filler metal is permitted.
	$\phi = 0.80$		See		
Compression Column to Base Plate and column splices designed per 10.10.1.4(a)	Compressive stress need not be considered in design of welds joining the parts.				
Compression Connections of members designed to bear other than columns as described in 10.10.1.4(b)	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See 10.10.4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	0.60 F_{EXX}	See 10.10.2.1.1	
Compression Connections not finished-to-bear	$\phi = 0.90$		See		
	$\phi = 0.80$		See		
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the				
Shear	Base Governed by 10.10.4				
	$\phi =$	0.60 F_{EXX}		See	

TABLE 10.10.2.5 (Contd.): Available Strength of Welded Joints, N

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength	Effective Area	Required Filler Metal Strength Level ^{[a][b]}	
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS						
Shear	Base	Governed by 10.10.4		Filler metal with a strength level equal to or less than matching filler metal is permitted.		
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}^{[d]}$ 10.10.2.2.1			
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.					
PLUG AND SLOT WELDS						
Shear Parallel to faying surface on the effective	Base	Governed by 10.10.4		Filler metal with a strength level equal to or less than matching filler metal is permitted.		
		$\phi = 0.60$	$F_{EXX}^{[d]}$			

For matching weld metal see AWS D1.1, Section 3.3.

Filler metal with a strength level one strength level greater than matching is permitted.

Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, $\phi = 0.80, \Omega = 1.88$ and $0.60 F_{EXX}$ as the nominal strength.

For the base metal

$$R_n = F_{BM} A_{BM} \quad (10.10.2.2)$$

For the weld metal

$$R_n = F_w A_w \quad (10.10.2.3)$$

Where

F_{BM} = nominal strength of the base metal per unit area, MPa

F_w = nominal strength of the weld metal per unit area, MPa

A_{BM} = cross-sectional area of the base metal, mm²

A_w = effective area of the weld, mm²

The values of ϕ , Ω , F_{BM} , and F_w and limitations thereon are given in Table 10.10.2.5.

Alternatively, for *fillet welds* loaded in-plane the *design strength*, ϕR_n and the allowable strength, R_n / Ω , of welds is permitted to be determined as follows:

$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

For a linear weld group loaded in-plane through the center of gravity

$$R_n = F_w A_w \quad (10.10.2.4)$$

where

$$F_w = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta) \quad (10.10.2.5)$$

And

F_{EXX} = electrode classification number, MPa

θ = angle of loading measured from the weld longitudinal axis, degrees

A_w = effective area of the weld, mm²

For weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method, the components of the *nominal strength*, R_{nx} and R_{ny} , are permitted to be determined as follows:

$$R_{nx} = \sum F_{wix} A_{wi} \quad R_{ny} = \sum F_{wiy} A_{wi} \quad (10.10.2.6)$$

where

A_{wi} = effective area of weld throat of any i th weld element, mm^2

$$F_{wi} = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta) f(p) \quad (10.10.2.7)$$

$$f(p) = [p(1.9 - 0.9p)]^{0.3} \quad (10.10.2.8)$$

F_{wi} = nominal stress in any i th weld element, MPa

F_{wix} = x component of stress, F_{wi}

F_{wiy} = y component of stress, F_{wi}

p = Δ_i / Δ_m , ratio of element i deformation to its deformation at maximum stress

w = weld leg size, mm

r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u / r_i ratio, mm

Δ_i = deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i , mm = $r_i \Delta_u / r_{crit}$

Δ_m = $0.209(\theta + 2)^{-0.32} w$, deformation of weld element at maximum stress, mm

Δ_u = $1.087(\theta + 6)^{-0.65} w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm

For *fillet weld* groups concentrically loaded and consisting of elements that

are oriented both longitudinally and transversely to the direction of applied *load*, the combined strength, R_n , of the fillet weld group shall be determined as the greater of

$$R_n = R_{wl} + R_{wt} \quad (10.10.2.9a)$$

or

$$R_n = 0.85 R_{wl} + 1.5 R_{wt} \quad (10.10.2.9b)$$

where

R_{wl} = the total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 10.10.2.5, N

R_{wt} = the total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 10.10.2.5 without the alternate in Section 10.10.2.4(a), N

10.10.2.5 Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single *joint*, the strength of each shall be separately computed with reference to the axis of the group in order to determine the strength of the combination.

10.10.2.6 Filler Metal Requirements

The choice of electrode for use with *complete-joint-penetration groove welds* subject to tension normal to the effective area shall comply with the requirements for matching *filler metals* given in AWS D1.1.

Filler metal with a specified *Charpy V-Notch* (CVN) toughness of 27.12 N-m (27 J) at 4°C shall be used in the following joints:

Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the *nominal strength* and *resistance factor* or *safety factor* as applicable for a PJP weld.

Complete-joint-penetration groove welded splices subject to tension normal to the effective area in heavy sections as defined in 10.1.3.1c and 10.1.3.1d.

10.10.2.7 Mixed Weld Metal

When Charpy V-Notch toughness is specified, the process consumables for all *weld metal*, tack welds, root pass and subsequent passes deposited in a *joint* shall be compatible to ensure notch-tough composite weld metal.

10.10.3 Bolts and Threaded Parts

10.10.3.1 High-Strength Bolts

Use of high-strength bolts shall conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, hereafter referred to as the RCSC Specification, as approved by the Research Council on Structural Connections, except as otherwise provided in this Specification.

When assembled, all *joint* surfaces, including those adjacent to the washers, shall be free of scale, except tight *mill scale*. All ASTM A325 or A325M and A490 or A490M bolts shall be tightened to a bolt tension not less than that given in Table 10.10.3.1, except as noted below. Except as permitted below, installation shall be assured by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench or alternative design bolt.

Bolts are permitted to be installed to only the snug-tight condition when used in *bearing-type connections*.

tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or *fatigue* due to vibration or *load* fluctuations are not design considerations.

TABLE 10.10.3.1 : Minimum Bolt Pretension, kN*

Bolt Size, mm	A325M Bolts	A490M Bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334

* Equal to 0.70 times the minimum *tensile strength* of bolts, rounded off to nearest kN, as specified in ASTM

The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When ASTM A490 or A490M bolts over 25 mm in diameter are used in slotted or oversized holes in external plies, a single hardened washer conforming to ASTM F436, except with 8 mm minimum thickness, shall be used in lieu of the standard washer.

In *slip-critical connections* in which the direction of loading is toward an edge of a connected part, adequate available bearing strength shall be provided based upon the applicable requirements of Section 10.10.3.10.

When bolt requirements cannot be provided by ASTM A325 and A325M, F1852, or A490 and A490M bolts because of requirements for lengths exceeding 12 diameters or diameters exceeding 38 mm, bolts or threaded rods conforming to ASTM A354 Gr. BC, A354 Gr. BD, or A449 are permitted to be used in accordance with the provisions for threaded rods in Table 10.10.3.2.

When ASTM A354 Gr. BC, A354 Gr. BD, or A449 bolts and threaded rods are used in slip-critical connections, the bolt geometry including the head and nut(s) shall be equal to or (if larger in diameter) proportional to that provided by ASTM A325 and A325M, or ASTM A490 and A490M bolts. Installation shall comply with all applicable requirements of the RCSC Specification with modifications as required for the increased diameter and/or length to provide the design pretension.

10.10.3.2 Size and Use of Holes

The maximum sizes of holes for bolts are given in Table 10.10.3.3, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in *column* base details.

Standard holes or *short-slotted holes* transverse to the direction of the *load* shall be provided in accordance with the provisions of this specification, unless over-sized holes, short-slotted holes parallel to the load or *long-slotted holes* are approved by the *engineer of record*. Finger *shims* up to 6 mm are permitted in *slip-critical*

connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in *bearing-type connections*. Hardened washers shall be installed over oversized holes in an outer ply.

Short-slotted holes are permitted in any or all plies of slip-critical or bearing-type connections. The slots are permitted without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual *faying surface*. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 8 mm thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

10.10.3.3 Minimum Spacing

The distance between centers of standard, oversized, or slotted holes, shall not be less than $2\frac{2}{3}$ times the nominal diameter, d , of the *fastener*; a distance of $3d$ is preferred.

10.10.3.4 Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table 10.10.3.4, or as required in Section 10.10.3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment C_2 from Table 10.10.3.5.

TABLE 10.10.3.2: Nominal Stress of Fasteners and Threaded Parts, MPa

Description of Fasteners	Nominal Tensile Stress, F_{nt} , MPa	Nominal Shear Stress in Bearing-Type Connections, F_{nv} , MPa
A307 bolts	310 ^{[a][b]}	165 ^{[b] [c] [f]}
A325 or A325M bolts, when threads are not excluded from shear planes	620 ^[e]	330 ^[f]
A325 or A325M bolts, when threads are excluded from shear planes	620 ^[e]	414 ^[f]
A490 or A490M bolts, when threads are not excluded from shear planes	780 ^[e]	414 ^[f]
A490 or A490M bolts, when threads are excluded from shear planes	780 ^[e]	520 ^[f]
Threaded parts meeting the requirements of Section 10.1.3.4, when threads are not excluded from shear planes	$0.75 F_u$ ^{[a][d]}	$0.40 F_u$
Threaded parts meeting the requirements of Section 10.1.3.4, when threads are excluded from shear planes	$0.75 F_u$ ^{[a][d]}	$0.50 F_u$

- [a] Subject to the requirements of Section 10.17.
- [b] For A307 bolts the tabulated values shall be reduced by 1 percent for each 2 mm over 5 diameters of length in the grip.
- [c] Threads permitted in shear planes.
- [d] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_D , which shall be larger than the nominal body area of the rod before upsetting times F_y .

TABLE 10.10.3.3: Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27 [a]	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75

[a] Clearance provided allows the use of a 25mm. bolt if desirable.

TABLE 10.10.3.4: Minimum Edge Distance,^[a] mm, from Center of Standard Hole^[b] to Edge of Connected Part

Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of
		Plates, Shapes or Bars, or Thermally Cut Edges [c]
16	28	22
20	34	26
22	38 [d]	28
24	42 [d]	30
27	48	34

[a] Lesser edge distances are permitted to be used provided provisions of Section 10.10.3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see Table 10.10.3.5.

[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where required strength does not exceed 25 percent of the maximum strength in the element.

TABLE 10.10.3.5
Values of Edge Distance Increment C_2 , mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes			Long Axis Parallel to Edge
		Long Axis Perpendicular to Edge		Long Slots ^[a]	
		Short Slots			
<22	2	3	0.75d	0	
24	3	3			
≥27	3	5			

^[a]When length of slot is less than maximum allowable (see Table 10.10.3.3), C₂ is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

^[a] When length of slot is less than maximum allowable (see Table 10.10.3.3), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

10.10.3.5 Maximum Spacing and Edge Distance

The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 150 mm. The longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a

For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 305 mm.

For unpainted members of *weathering steel* subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 180 mm.

10.10.3.6 Tension and Shear Strength of Bolts and Threaded Parts

The *design tension* or *shear strength*, ϕR_n and the *allowable tension* or *shear strength*, R_n/Ω of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the *limit states* of *tensile rupture* and *shear rupture* as follows:

$$R_n = F_n A_b \quad (10.10.3.1)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

F_n = nominal tensile stress F_{nt} , or shear stress, F_{nv} from Table 10.10.3.2, MPa

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d, Table 10.10.3.2, mm²)

The required *tensile strength* shall include any tension resulting from *prying action* produced by deformation of the connected parts.

10.10.3.7 Combined Tension and Shear in Bearing-Type Connection

The *available tensile strength* of a bolt subjected to combined tension and shear shall be determined according to the *limit states of tension* and *shear rupture* as follows:

$$R_n = F'_{nt} A_b \quad (10.10.3.2)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

F'_{nt} = nominal tensile stress modified to include the effects of shearing stress, MPa

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \text{ (LRFD)} \quad (10.10.3.3a)$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \text{ (ASD)} \quad (10.10.3.3b)$$

F_{nt} = nominal tensile stress from Table 10.10.3.2, MPa

F_{nv} = nominal shear stress from Table 10.10.3.2, MPa

f_v = the required shear stress, MPa

The available shear stress of the *fastener* shall equal or exceed the required shear strength per unit area, f_v .

10.10.3.7 High-Strength Bolts in Slip-Critical Connections

High-strength bolts in *slip-critical* connections are permitted to be designed to prevent *slip* either as a serviceability limit state or at the required strength limit state. The connection must also be checked for shear strength in accordance with Sections 10.10.3.6 and 10.10.3.7 and bearing strength in accordance with Sections 10.10.3.1 and 10.10.3.10.

Slip-critical connections shall be designed as follows, unless otherwise designated by the *engineer of record*. Connections with standard holes or slots transverse to the direction of the load shall be designed for slip as a serviceability limit state. Connections with oversized holes or slots parallel to the direction of the load shall be designed to prevent slip at the required strength level.

The design slip resistance, ϕR_n and the allowable slip resistance, R_n/Ω , shall be determined for the *limit state* of slip as follows:

$$R_n = \mu D_u h_{sc} T_b N_s \quad (10.10.3.4)$$

For connections in which prevention of slip is a serviceability limit state

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

For connections designed to prevent slip at the required strength level

$$\phi = 0.85 \text{ (LRFD)} \quad \Omega = 1.76 \text{ (ASD)}$$

Where

μ = mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests

= 0.35 for Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel and hot-dipped galvanized and roughened surfaces)

= 0.50 for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

$D_u = 1.13$; a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension. The use of other values may be approved by the engineer of record.

h_{sc} = hole factor determined as follows:

- a) For standard size holes $h_{sc} = 1.00$
- c) For oversized and short-slotted holes $h_{sc} = 0.85$
- d) For long-slotted holes $h_{sc} = 0.70$

N_s = number of slip planes

T_b = minimum fastener tension given in Table 10.10.3.1, kN

10.10.3.8 Combined Tension and Shear in Slip-Critical Connections

When a *slip-critical connection* is subjected to an applied tension that reduces the net clamping *force*, the available *slip* resistance per bolt, from Section 10.10.3.8, shall be multiplied by the factor, k_s , as follows:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} \quad \text{(LRFD)} \quad (10.10.3.5a)$$

$$k_s = 1 - \frac{1.5 T_a}{D_u T_b N_b} \quad \text{(ASD)} \quad (10.10.3.5b)$$

where

N_b = number of bolts carrying the applied tension

T_a = tension force due to *ASD load combinations*, kN

T_b = minimum *fastener* tension given in Table 10.10.3.1, kN

T_u = tension *force* due to *LRFD load combinations*, kN

10.10.3.10 Bearing Strength at Bolt Holes

The available bearing strength, ϕR_n and R_n/Ω , at bolt holes shall be determined for the *limit state* of bearing as follows:

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

For a bolt in a connection with standard, oversized, and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:

- a) When deformation at the bolt hole at service load is a design consideration

$$R_n = 1.2 L_c t F_u \leq 2.4 d t F_u \quad (10.10.3.6a)$$

- e) When deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5 L_c t F_u \leq 3.0 d t F_u \quad (10.10.3.6b)$$

For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$R_n = 1.0 L_c t F_u \leq 2.0 d t F_u \quad (10.10.3.6c)$$

For connections made using bolts that pass completely through an unstiffened box member or *HSS* see Section 10.10.7 and Equation 10.10.7.1,

where

d = nominal bolt diameter, mm

F_u = specified minimum tensile strength of the connected material, MPa

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm

t = thickness of connected material, mm

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

Bearing strength shall be checked for both bearing-type and slip-critical connections. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section 10.10.3.2.

10.10.3.9 Special Fasteners

The *nominal strength* of special fasteners other than the bolts presented in Table 10.10.3.2 shall be verified by tests.

10.10.3.10 Tension Fasteners

When bolts or other fasteners in tension are attached to an unstiffened box or *HSS* wall, the strength of the wall shall be determined by rational analysis.

10.10.4 Affected Elements of Members and Connecting Elements

This section applies to elements of members at *connections* and connecting elements, such as plates, gussets, angles, and brackets.

10.10.4.1 Strength of Elements in Tension

The *design strength*, ϕR_n and the *allowable strength*, R_n / Ω , of affected and connecting elements loaded in tension shall be the lower value obtained according to the *limit states* of *tensile yielding* and *tensile rupture*.

For tensile yielding of connecting elements:

$$R_n = F_y A_g \quad (10.10.4.1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

For tensile rupture of connecting elements:

$$R_n = F_u A_e \quad (10.10.4.2)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

A_e = *effective net area* as defined in Section 10.4.3.3, mm²; for bolted

splice plates, $A_e = A_n \leq 0.85 A_g$

10.10.4.2 Strength of Elements in Shear

The available shear yield strength of affected and connecting elements in shear shall be the lower value obtained according to the *limit states* of *shear yielding* and *shear rupture*:

For shear yielding of the element:

$$R_n = 0.60 F_y A_g \quad (10.10.4.3)$$

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \quad (10.10.4.4)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

A_{nv} = net area subject to shear, mm²

10.10.4.3 Block Shear Strength

The *available strength* for the *limit state of block shear rupture* along a shear failure path or path(s) and a perpendicular tension failure path shall be taken as

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \quad (10.10.4.5)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

where

A_{gv} = gross area subject to shear, mm²

A_{nt} = *net area* subject to tension, mm²

A_{nv} = net area subject to shear, mm²

Where the tension *stress* is uniform, $U_{bs} = 1$; where the tension stress is non-uniform, $U_{bs} = 0.5$.

10.10.4.3 Strength of Elements in Compression

The available strength of connecting elements in compression for the *limit states of yielding* and *buckling* shall be determined as follows.

For $KL/r \leq 25$

$$P_n = F_yA_g \quad (10.10.4.6)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

For $KL/r > 25$ the provisions of Section 10.5 apply.

10.10.5 Fillers

In welded construction, any *filler* 6 mm or more in thickness 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. The welds joining 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 less than 6 mm thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When a bolt that carries load passes through fillers that are equal to or less than 6 mm thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 6 mm thick, one of the following requirements shall apply:

For fillers that are equal to or less than 19 mm thick, the shear strength of the bolts shall be multiplied by the factor $[1 - 0.0154(t - 6)]$, where t is the total thickness of the fillers up to 19 mm;

The fillers shall be extended beyond the *joint* and the filler extension shall be secured with enough bolts to uniformly distribute the total *force* in the connected element over the combined cross section of the connected element and the fillers;

The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

The joint shall be designed to prevent *slip* at required strength levels in accordance with Section 10.10.3.8.

10.10.6 Splices

Groove-welded *splices* in *plate girders* and *beams* shall develop the *nominal strength* of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

10.10.7 Bearing Strength

The design bearing strength, ϕR_n and the allowable bearing strength, R_n/Ω , of surfaces in contact shall be determined for the limit state of bearing (local compressive yielding) as follows:

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

The nominal bearing strength, R_n , is defined as follows for the various types of bearing:

For milled surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners:

$$R_n = 1.8F_y A_{pb} \quad (10.10.7.1)$$

where

F_y = specified minimum yield stress, MPa

A_{pb} = projected bearing area, mm²

For expansion rollers and rockers:

If $d \leq 635$ mm

$$R_n = 1.2(F_y - 90)ld/20 \quad (10.10.7.2)$$

If $d > 635$ mm

$$R_n = 30.2(F_y - 90)l\sqrt{d}/20 \quad (10.10.7.3)$$

where

d = diameter, mm

l = length of bearing, mm

10.10.8 Column Bases and Bearing on Concrete

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, the design bearing strength, $\phi_c P_p$, and the allowable bearing strength, P_p / Ω_c , for the *limit state of concrete crushing* are permitted to be taken as follows:

$\phi_c = 0.60$ (LRFD) $\Omega_c = 2.50$ (ASD)

The nominal bearing strength, P_p , is determined as follows:

On the full area of a concrete support:

$$P_p = 0.8f'_c A_1 \quad (10.10.8.1)$$

On less than the full area of a concrete support:

$$P_p = 0.8f'_c A_1 \sqrt{A_2/A_1} \leq 1.7f'_c A_1 \quad (10.10.8.2)$$

where

A_1 = area of steel concentrically bearing on a concrete support, mm²

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm²

10.10.9 Anchor Rods and Embedments

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment that may result from load combinations stipulated in Section 10.2.2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table 10.10.3.2.

Larger oversized and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using structural or plate washers to bridge the hole.

When horizontal forces are present at *column* bases, these forces should, where possible, be resisted by bearing against concrete elements or by shear friction between the column base plate and the foundation. When anchor rods are designed to resist horizontal force the base plate hole size, the anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

10.10.10 Flanges and Webs with Concentrated Forces

This section applies to *single-* and *double-concentrated forces* applied normal to the flange(s) of wide flange sections and similar built-up shapes. A single- concentrated force can be either tensile or compressive. Double-

concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the *required strength* exceeds the *available strength* as determined for the *limit states* listed in this section, *stiffeners* and/or *doublers* shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable *limit state*. Stiffeners shall also meet the design requirements in Section 10.10.10.8. Doublers shall also meet the design requirement in Section 10.10.10.9.

Stiffeners are required at *unframed ends* of *beams* in accordance with the requirements of Section 10.10.10.7.

10.10.10.1 Flange Local Bending

This section applies to tensile *single-concentrated forces* and the tensile component of *double-concentrated forces*.

The *design strength*, ϕR_n and the *allowable strength*, R_n / Ω , for the *limit state* of flange *local bending* shall be determined as follows:

$$R_n = 6.25 t_f^2 F_{yf} \quad (10.10.10.1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where

F_{yf} = specified minimum yield stress of the flange, MPa

t_f = thickness of the loaded flange, mm

If the length of loading across the member flange is less than $0.15 b_f$, where b_f is the member flange width, Equation 10.10.10.1 need not be checked.

When the concentrated *force* to be resisted is applied at a distance from the member end that is less than $10 t_f$, R_n shall be reduced by 50 percent.

When required, a pair of *transverse stiffeners* shall be provided.

10.10.10.2 Web Local Yielding

This section applies to *single-concentrated forces* and both components of *double-concentrated forces*.

The *available strength* for the *limit state* of web *local yielding* shall be determined as follows:

$$\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

The *nominal strength*, R_n , shall be determined as follows:

When the concentrated *force* to be resisted is applied at a distance from the member end that is greater than the depth of the member d ,

$$R_n = (5k + N) F_{yw} t_w \quad (10.10.10.2)$$

When the concentrated *force* to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d ,

$$R_n = (2.5k + N) F_{yw} t_w \quad (10.10.10.3)$$

where

k = distance from outer face of the flange to the web toe of the fillet, mm

F_{yw} = specified minimum yield stress of the web, MPa

N = length of bearing (not less than k for end beam reactions), mm

t_w = web thickness, mm

When required, a pair of *transverse stiffeners* or a *doubler* plate shall be provided.

10.10.10.3 Web Crippling

This section applies to compressive *single-concentrated forces* or the compressive component of *double-concentrated forces*.

The *available strength* for the *limit state* of web *local crippling* shall be determined as follows:

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

The *nominal strength*, R_n , shall be determined as follows:

When the concentrated compressive *force* to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$:

$$R_n = 0.80t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (10.10.10.4)$$

When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$:

For $N/d \leq 0.2$

$$R_n = 0.40t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (10.10.10.5a)$$

For $N/d > 0.2$

$$R_n = 0.40t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} \quad (10.10.10.5b)$$

where

d = overall depth of the member, mm

t_f = flange thickness, mm

When required, a *transverse stiffener* or pair of transverse stiffeners, or a *doubler* plate extending at least one-half the depth of the web shall be provided.

10.10.10.4 Web Sidesway Buckling

This Section applies only to compressive *single-concentrated forces* applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated *force*.

The *available strength* of the web shall be determined as follows:

$$\phi = 0.85 \text{ (LRFD)} \quad \Omega = 1.76 \text{ (ASD)}$$

The nominal strength, R_n , for the *limit state* of *web sidesway buckling* shall be determined as follows:

If the compression flange is restrained against rotation:

For $(h/t_w)/(l/b_f) \leq 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[1 + 0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \quad (10.10.10.6)$$

For $(h/t_w)/(l/b_f) > 2.3$, the limit state of web sidesway buckling does not apply.

When the *required strength* of the web exceeds the available strength, local *lateral bracing* shall be provided at the tension flange or either a pair of *transverse stiffeners* or a *doubler* plate shall be provided.

If the compression flange is not restrained against rotation

For $(h/t_w)/(l/b_f) \leq 1.7$

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \quad (10.10.10.7)$$

For $(h/t_w)/(l/b_f) > 1.7$ the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local *lateral bracing* shall be provided at both flanges at the point of application of the concentrated forces.

In Equations 10.10.10.6 and 10.10.10.7, the following definitions apply:

b_f = flange width, mm

$C_r = 6.62 \times 10^6$ MPa when $M_u < M_y$ (LRFD) or $1.5 M_u < M_y$ (ASD) at the location of the force

$= 3.31 \times 10^6$ MPa when $M_u \geq M_y$ (LRFD) or $1.5 M_a \geq M_y$ (ASD) at the location of the force

h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of *fasteners* or the clear distance between flanges when welds are used for built-up shapes, mm

l = largest laterally *unbraced length* along either flange at the point of load, mm

t_f = flange thickness, mm

t_w = web thickness, mm

10.10.10.5 Web Compression Buckling

This Section applies to a pair of compressive *single-concentrated forces* or the compressive components in a pair of *double-concentrated forces*, applied at both flanges of a member at the same location.

The *available strength* for the *limit state* of web *local buckling* shall be determined as follows:

$$R_n = \frac{24t_w^3 \sqrt{EF_{yw}}}{h} \quad (10.10.10.8)$$

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

When the pair of concentrated compressive *forces* to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50 percent.

When required, a single *transverse stiffener*, a pair of transverse stiffeners, or a *doubler* plate extending the full depth of the web shall be provided.

10.10.10.6 Web Panel Zone Shear

This section applies to *double-concentrated forces* applied to one or both flanges of a member at the same location.

The *available strength* of the web *panel zone* for the *limit state* of *shear yielding* shall be determined as follows:

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

The *nominal strength*, R_n , shall be determined as follows:

When the effect of panel-zone deformation on frame *stability* is not considered in the analysis:

For $P_r \leq 0.4P_c$

$$R_n = 0.60F_y d_c t_w \quad (10.10.10.9)$$

For $P_r > 0.4P_c$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c} \right) \quad (10.10.10.10)$$

When frame stability, including plastic panel-zone deformation, is considered in the analysis:

For $P_r \leq 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w} \right) \quad (10.10.10.11)$$

For $P_r > 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w} \right) \left(1.9 - \frac{1.2P_r}{P_c} \right) \quad (10.10.10.12)$$

In Equations 10.10.10.9 through 10.10.10.12, the following definitions apply:

A = *column* cross-sectional area, mm²

b_{cf} = width of column flange, mm

d_b = *beam* depth, mm

d_c = column depth, mm

F_y = *specified minimum yield stress* of the column web, MPa

$P_c = P_y$, N (LRFD)

$P_c = 0.6 P_y$, N (ASD)

P_r = required strength, N

P_y = $F_y A$, axial yield strength of the column, N

t_{cf} = thickness of the column flange, mm

t_w = column web thickness, mm

When required, *doubler* plate(s) or a pair of *diagonal stiffeners* shall be provided within the boundaries of the rigid connection whose webs lie in a common plane.

See Section 10.10.10.9 for doubler plate design requirements.

10.10.10.7 Unframed Ends of Beams and Girders

At *unframed ends* of *beams* and *girders* not otherwise restrained against rotation about their longitudinal axes, a pair of *transverse stiffeners*, extending the full depth of the web, shall be provided.

10.10.10.8 Additional Stiffeners Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated *forces* shall be designed in accordance with the requirements of Section 10.4 and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the *required strength* and available *limit state* strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces shall be designed in accordance with the requirements in Sections 10.5.6.2 and 10.10.4.4 and shall either bear on or be welded to the loaded flange and welded to the web. The welds to the flange shall be sized for the difference between the required strength and the applicable limit state strength. The weld to the web shall be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener. For fitted bearing stiffeners, see Section 10.10.7.

Transverse full depth bearing stiffeners for compressive forces applied to a *beam* or *plate girder* flange(s) shall be designed as axially compressed members (*columns*) in accordance with the requirements of Sections 10.5.6.2 and 10.10.4.4.

The member properties shall be determined using an *effective length* of $0.75h$ and a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members. The weld connecting full depth bearing stiffeners to the web shall be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and *diagonal stiffeners* shall comply with the following additional criteria:

The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.

The thickness of a *stiffener* shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated *load*, and greater than or equal to the width divided by 15.

Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in 10.10.10.5 and 10.10.10.7.

10.10.10.9 Additional doubler Plate Requirements for Concentrated Forces

Doubler plates required for compression strength shall be designed in accordance with the requirements of Section 10.5.

Doubler plates required for tensile strength shall be designed in accordance with the requirements of Section 10.4.

Doubler plates required for shear strength (see Section 10.10.10.6) shall be designed in accordance with the provisions of Section 10.7.

In addition, doubler plates shall comply with the following criteria:

The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

10.11 Design of HSS and Box Member Connections

This section covers member strength design considerations pertaining to connections to HSS members and box sections of uniform wall thickness. See also Section 10.10 for additional requirements for bolting to HSS.

10.11.1 Concentrated Forces on HSS

10.11.1.1 Definitions of Parameters

B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, mm.

B_p = width of plate, measured 90 degrees to the plane of the connection, mm.

D = outside diameter of round HSS member, mm.

F_y = specified minimum yield stress of HSS member material, MPa.

F_{yp} = specified minimum yield stress of plate, MPa.

F_u = specified minimum tensile strength of HSS material, MPa.

H = overall height of rectangular HSS member, measured in the plane of the connection, mm.

N = bearing length of the load, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of loaded cap plates), mm.

t = design wall thickness of HSS member, mm.

t_p = thickness of plate, mm.

10.11.1.2 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

Strength: $F_y \leq 360$ MPa. for HSS

Ductility: $F_y/F_u \leq 0.8$ for HSS

Other limits apply for specific criteria

10.11.1.3 Concentrated Force Distributed Transversely

10.11.1.3.1 Criterion for Round HSS

When a concentrated force is distributed transversely to the axis of the HSS the design strength, ΦR_n and the allowable strength, R_n/Ω , for the limit state of local yielding shall be determined as follows:

$$R_n = F_y t^2 [5.5/(1 - 0.81B_p/D)] Q_f \quad (10.11.1.1)$$

$$\Phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation 10.11.2.1. Additional limits of applicability are

$$0.2 < B_p / D \leq 1.0$$

$D/t \leq 50$ for T-connections and $D/t \leq 40$ for cross-connections

10.11.1.3.2 Criterion for Rectangular HSS

When a concentrated force is distributed transversely to the axis of the HSS the design strength, ΦR_n and the allowable strength, R_n/Ω , shall be the lowest value according to the limit states of local yielding due to uneven load distribution, shear yielding (punching) and sidewall strength.

Additional limits of applicability are

$$0.25 < B_p / B \leq 1.0$$

B/t for the loaded HSS wall ≤ 35

For the limit state of local yielding due to uneven load distribution in the loaded plate,

$$R_n = [10/F_y t/(B/t)] B_p \leq F_{yp} t_p B_p \quad (10.11.1.2)$$

$$\Phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

For the limit state of shear yielding (punching),

$$R_n = 0.6F_y t [2t_p + 2B_{ep}] \quad (10.11.1.3)$$

$$\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

Where

$$B_{ep} = 10B_p / (B/t) \leq B_p$$

This limit state need not be checked when $B_p > (B - 2t)$, nor when $B_p < 0.85 B$.

For the limit state of sidewall under tension loading, the available strength shall be taken as the strength for sidewall local yielding. For the limit state of sidewall under compression loading, available strength shall be taken as the lowest value obtained according to the limit states of sidewall local yielding, sidewall local crippling and sidewall local buckling.

This limit state need not be checked unless the chord member and branch member (connecting element) have the same width ($\beta = 1.0$).

For the limit state of sidewall local yielding,

$$R_n = 2F_y t [5k + N] \quad (10.11.1.4)$$

$$\phi = 1.0 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$$

where

k = outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, mm.

For the limit state of sidewall local crippling, in T-connections,

$$R_n = 1.6t^2 [1 + 3N/(H - 3t)] (EF_y)^{0.5} Q_f \quad (10.11.1.5)$$

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.0 \text{ (ASD)}$$

where Q_f is given by Equation 10.11.2.10.

For the limit state of sidewall local buckling in cross-connections,

$$R_n = [48t^3 / (H - 3t)] (EF_y)^{0.5} Q_f \quad (10.11.1.6)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation 10.11.2.10

The nonuniformity of load transfer along the line of weld, due to the flexibility of the HSS wall in a transverse plate-to-HSS connection, shall be considered in proportioning such welds. This requirement can be satisfied by limiting the total effective weld length, L_e , of groove and fillet welds to rectangular HSS as follows:

$$L_e = 2[10/(B/t)] [(F_t t) / (F_{yp} t_p)] B_p \leq 2B_p \quad (10.11.1.7)$$

where

L_e = total effective weld length for welds on both sides of the transverse plate, mm.

In lieu of Equation 10.11.17, this requirement may be satisfied by other rational approaches.

10.11.1.4 Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width and Acting Perpendicular to the HSS Axis

When a concentrated *force* is distributed longitudinally along the axis of the HSS at the center of the HSS diameter or width, and also acts perpendicular to the axis direction of the HSS (or has a component perpendicular to the axis direction of the HSS), the *design strength*, ϕR_n and the *allowable strength*, R_n / Ω , perpendicular to the HSS axis shall be determined for the *limit state* of *chord plastification* as follows.

10.11.1.4.1 Criterion for Round HSS

An additional limit of applicability is:

$D/t \leq 50$ for T-connections and $D/t \leq 40$ for cross-connections

$$R_n = 5.5F_y t^2 (1 + 0.25N/D) Q_f \quad (10.11.1.8)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation 10.11.2.1.

10.11.1.4.2 Criterion for Rectangular HSS

An additional limit of applicability is:

B/t for the loaded HSS wall ≤ 40

$$R_n = [F_y t^2 / (1 - t_p/B)] [2N/B + 4(1 - t_p/B)^{0.5} Q_f] \quad (10.11.1.9)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

where

$$Q_f = (1 - U^2)^{0.5}$$

U is given by Equation 10.11.2.12

10.11.1.5 Concentrated Force Distributed Longitudinally at the Center of the HSS Width and Acting Parallel to the HSS Axis

When a concentrated *force* is distributed longitudinally along the axis of a rectangular *HSS* and also acts parallel but eccentric to the axis direction of the member, the *connection* shall be verified as follows:

$$F_{yp} t_p \leq F_u t \quad (10.11.1.10)$$

10.11.1.6 Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate

When a concentrated *force* acts on the end of a capped *HSS* and the force is in the direction of the *HSS* axis, the *design strength*, ϕR_n and the allowable strength, R_n/Ω , shall be determined for the *limit states* of wall local *yielding* (due to tensile or compressive *forces*) and wall *local crippling* (due to compressive forces only), with consideration for shear lag, as follows.

If $(5t_p + N) \geq B$, the *available strength* of the *HSS* is computed by summing the contributions of all four *HSS* walls.

If $(5t_p + N) < B$, the available strength of the *HSS* is computed by summing the contributions of the two walls into which the *load* is distributed.

For the limit state of wall local yielding, for one wall,

$$R_n = F_y t [5t_p + N] \leq B F_y t \quad (10.11.1.11)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

For the limit state of wall local crippling, for one wall,

$$R_n = 0.8 t^2 [1 + (6N/B)(t/t_p)^{1.5}] [E F_y t_p / t]^{0.5} \quad (10.11.1.12)$$

$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

10.11.2 HSS-To-HSS Truss Connections

HSS-to-HSS truss *connections* are defined as connections that consist of one or more *branch members* that are directly welded to a continuous chord that passes through the connection and shall be classified as follows:

- When the punching load ($P_r \sin \theta$) in a branch member is equilibrated by *beam* shear in the *chord member*, the connection shall be classified as a *T-connection* when the branch is perpendicular to the chord and a *Y-connection* otherwise.
- When the punching load ($P_r \sin \theta$) in a branch member is essentially equilibrated (within 20 percent) by *loads* in other branch member(s) on the same side of the connection, the connection shall be classified as a *K-connection*. The relevant gap is between the primary branch members whose loads equilibrate. An *N-connection* can be considered as a type of *K-connection*.
- When the punching load ($P_r \sin \theta$) is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection shall be classified as a *cross-connection*.
- When a connection has more than two primary branch members or branch members in more than one plane, the connection shall be classified as a general or multiplanar connection.

- e) When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the *nominal strength* shall be determined by interpolation on the proportion of each in total.

For the purposes of this Specification, the centerlines of branch members and chord members shall lie in a common plane. Rectangular HSS connections are further limited to have all members oriented with walls parallel to the plane. For trusses that are made with HSS that are connected by welding branch members to chord members, eccentricities within the limits of applicability are permitted without consideration of the resulting moments for the design of the connection.

10.11.2.1 Definitions of Parameters

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, mm.

B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, mm.

D = outside diameter of round HSS main member, mm.

D_b = outside diameter of round HSS branch member, mm.

E = eccentricity in a truss connection, positive being away from the branches, mm.

F_y = specified minimum yield stress of HSS main member material, MPa.

F_{yb} = specified minimum yield stress of HSS branch member material, MPa.

F_u = specified minimum tensile strength of HSS material, MPa.

G = gap between toes of branch members in a gapped K-connection, neglecting the welds, mm.

H = overall height of rectangular HSS main member, measured in the plane of the connection, mm.

H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, mm.

t = design wall thickness of HSS main member, mm.

t_b = design wall thickness of HSS branch member, mm.

β = the width ratio; the ratio of branch diameter to chord diameter = D_b / D for round HSS; the ratio of overall branch width to chord width = B_b / B for rectangular HSS

β_{eff} = the *effective width* ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width

γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/(2t)$ for round HSS; the ratio of one-half the width to wall thickness = $B/(2t)$ for rectangular HSS

H = the *load* length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B , where $N = H_b / \sin\theta$

θ = acute angle between the branch and chord (degrees)

ξ = the gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord = g/B for rectangular HSS

10.11.2.2 Criteria for Round HSS

The interaction of stress due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \quad (10.11.2.1)$$

where U is the utilization ratio given by

$$U = |P_r / (A_g F_c) + M_r / (S F_c)| \quad (10.11.2.2)$$

and

P_r = required axial strength in chord, N; for K-connections, P_r is to be determined on the side of the *joint* that has the lower compression stress (lower U)

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6 F_y$, MPa.

10.11.2.2.1 Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits of applicability:

Joint eccentricity: $-0.55 D \leq e \leq 0.25 D$, where D is the chord diameter and e is positive away from the branches

Branch angle: $\theta \geq 30^\circ$

Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for *T*-, *Y*- and *K*-connections; less than or equal to 40 for *cross-connections*

Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50

Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05 E / F_y$

Width ratio: $0.2 < D_b / D \leq 1.0$ in general, and $0.4 \leq D_b / D \leq 1.0$ for gapped K-connections

If a *gap connection*: g greater than or equal to the sum of the branch wall thicknesses

If an *overlap connection*: $25\% \leq O_v \leq 100\%$, where $O_v = (q / p) \times 100\%$. P is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal diameter, the thicker) branch is a "thru member" connected directly to the chord.

Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch

Strength: $F_y \leq 360$ MPa. for chord and branches

Ductility: $F_y / F_u \leq 0.8$

10.11.2.2.2 Branches with Axial Loads in T-, Y- and Cross-Connections

For T- and Y- connections, the *design strength* of the branch ϕP_n or the *allowable strength* of the branch, P_n / Ω , shall be the lower value obtained according to the *limit states of chord plastification and shear yielding (punching)*.

For the limit state of chord plastification in T- and Y-connections,

$$P_n \sin \theta = F_y t^2 [3.1 + 15.6 \beta^2] \gamma^{0.2} Q_f \quad (10.11.2.3)$$

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For the limit state of shear yielding (punching),

$$P_n = 0.6 F_y t \pi D_b [(1 + \sin \theta) / 2 \sin^2 \theta] \quad (10.11.2.4)$$

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

For the limit state of chord plastification in cross-connections,

$$P_n \sin \theta = F_y t^2 [5.7 / (1 - 0.81 \beta)] Q_f \quad (10.11.2.5)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

10.11.2.2.3 Branches with Axial Loads in K-Connections

For K-connections, the design strength of the branch, ΦP_n and the allowable strength of the branch, P_n/Ω , shall be the lower value obtained according to the limit states of chord plastification for gapped and overlapped connections and shear yielding (punching) for gapped connections only.

For the limit state of chord plastification,

$$\Phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

For the compression branch:

$$P_n \sin \theta = F_y t^2 [2.0 + 11.33 D_b / D] Q_g Q_f \quad (10.11.2.6)$$

where D_b refers to the compression branch only, and

$$Q_g = \gamma^{0.2} \left[1 + \frac{0.024 \gamma^{1.2}}{e^{(\frac{0.5g}{t} - 1.33)} + 1} \right] \quad (10.11.2.7)$$

In gapped connections, g (measured along the crown of the chord neglecting weld dimensions) is positive. In overlapped connections, g is negative and equals q .

For the tension branch,

$$P_n \sin \theta = (P_n \sin \theta)_{\text{compression branch}} \quad (10.11.2.8)$$

For the limit state of shear yielding (punching) in gapped K-connections,

$$P_n = 0.6 F_y t \pi D_b [(1 + \sin \theta) / 2 \sin^2 \theta] \quad (10.11.2.9)$$

$$\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

10.11.2.3 Criteria for Rectangular HSS

The interaction of *stress* due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression in *T*-, *Y*-, and *cross-connections*,

$$Q_f = 1.3 - 0.4U/\beta \leq 1 \quad (10.11.2.10)$$

When the chord is in compression in gapped *K-connections*,

$$Q_f = 1.3 - 0.4U/\beta_{eff} \leq 1 \quad (10.11.2.11)$$

where U is the utilization ratio given by

$$U = |P_r / (A_g F_c) + M_r / (S F_c)| \quad (10.11.2.12)$$

and

P_r = required axial strength in chord, N. For gapped K-connections, P_r is to be determined on the side of the *joint* that has the higher compression stress (higher U).

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, N

$M_r = M_u$ = required flexural strength in chord, using *LRFD load combinations*, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_o$ = required axial strength in chord, using *ASD load combinations*, N.

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6 F_y$, MPa.

10.11.2.3.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits:

Joint eccentricity: $-0.55 H \leq e \leq 0.25H$, where H is the chord depth and e is positive away from the branches

Branch angle: $\theta \geq 30^\circ$

Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to 30 for overlapped K-connections

Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35

Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to $1.1(E/F_{yb})^{0.5}$ for overlapped K-connections

Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25 for T-, Y-, cross- and overlapped K-connections; greater than or equal to 0.35 for gapped K-connections

Aspect ratio: $0.5 \leq \text{ratio of depth to width} \leq 2.0$

Overlap: $25\% \leq O_v \leq 100\%$, where $O_v = (q / p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal width, the thicker) branch is a "thru member" connected directly to the chord

Branch width ratio for overlap connections: ratio of overall wall width of overlapping branch to overall wall width of overlapped branch greater than or equal to 0.75

Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch

Strength: $F_y \leq 360$ MPa. for chord and branches

Ductility: $F_y / F_u \leq 0.8$

Other limits apply for specific criteria

10.11.2.3.2 Branches with Axial Loads in T-,Y- and Cross-Connections

For T-, Y- and cross-connections, the *design strength* of the branch, ϕP_n or the *allowable strength* of the branch, P_n / Ω , shall be the lowest value obtained according to the *limit states* of *chord wall plastification*, *shear yielding (punching)*, *sidewall strength* and *local yielding* due to uneven load distribution. In addition to the limits of applicability in Section 10.11.2.3a, β shall not be less than 0.25.

For the limit state of chord wall plastification,

$$P_n \sin \theta = F_y t^2 [2\eta / (1 - \beta) + 4 / (1 - \beta)^{0.5}] Q_f \quad (10.11.2.13)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

This limit state need not be checked when $\beta > 0.85$.

For the limit state of shear yielding (punching),

$$P_n \sin \theta = 0.6 F_y t B [2\eta + 2\beta_{eop}] \quad (10.11.2.14)$$

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

In Equation 10.11.2.14, the effective outside punching parameter $\beta_{eop} = 5\beta / \gamma$ shall not exceed β .

This limit state need not be checked when $\beta > (1 - 1 / \gamma)$, nor when $\beta < 0.85$ and $B/t \geq 10$.

For the limit state of sidewall strength, the *available strength* for branches in tension shall be taken as the available strength for sidewall local yielding. For the limit state of sidewall strength, the available strength for branches in compression shall be taken as the lower of the strengths for sidewall local yielding and sidewall local crippling. For cross-connections with a branch angle less than 90 degrees, an additional check for chord sidewall shear failure must be made in accordance with Section 10.7.5.

This limit state need not be checked unless the chord member and branch member have the same width ($\beta = 1.0$)

For the limit state of local yielding,

$$P_n \sin \theta = 2F_y t [5k + N] \quad (10.11.2.15)$$

$\Phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

where

k = outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, mm.

N = bearing length of the *load*, parallel to the axis of the HSS main member, $H_b/\sin\theta$, mm.

For the limit state of sidewall local crippling, in T- and Y-connections,

$$P_n \sin \theta = 1.6t^2 [1 + 3N/(H - 3t)] (EF_y)^{0.5} Q_f \quad (10.11.2.16)$$

$\Phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

For the limit state of sidewall local crippling in cross-connections,

$$P_n \sin \theta = [48t^3/(H - 3t)] (EF_y)^{0.5} Q_f \quad (10.11.2.17)$$

$\Phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb} t_b [2H_b + 2b_{eoi} - 4t_b] \quad (10.11.2.18)$$

$\Phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

where

$$b_{eoi} = [10/(B/t)] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (10.11.2.19)$$

This limit state need not be checked when $\beta < 0.85$.

10.11.2.3.3 Branches with Axial Loads in Gapped K-Connections

For gapped K-connections, the design strength of the branch, ΦP_n or the allowable strength of the branch, P_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, *shear yielding* (punching), shear yielding and local yielding due to uneven load distribution. In addition to the limits of applicability in Section 10.11.2.3a, the following limits shall apply:

- $B_b/B \geq 0.1 + \gamma/50$
- $\beta_{eff} \geq 0.35$
- $\zeta \leq 0.5(1 - \beta_{eff})$
- Gap: g greater than or equal to the sum of the branch wall thicknesses
- The smaller $B_b > 0.63$ times the larger B_b

For the limit state of chord wall plastification,

$$P_n \sin \theta = F_y t^2 [9.8\beta_{eff}\gamma^{0.5}] Q_f \quad (10.11.2.20)$$

$\Phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For the limit state of shear yielding (punching),

$$P_n \sin \theta = 0.6F_y t B [2\eta + \beta + \beta_{eop}] \quad (10.11.2.21)$$

$\Phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

In the above equation, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β .

This limit state need only be checked if $B_b < (B - 2t)$ or the branch is not square.

For the limit state of shear yielding of the chord in the gap, available strength shall be checked in accordance with Section 10.7. This limit state need only be checked if the chord is not square.

For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb} t_b [2H_b + B_b + b_{eoi} - 4t_b] \quad (10.11.2.22)$$

$\Phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

where

$$b_{eoi} = [10/(B/t)] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (10.11.2.23)$$

This limit state need only be checked if the branch is not square or $B/t < 15$.

10.11.2.3.4 Branches with Axial Loads in Overlapped K-Connections

For overlapped K-connections, the design strength of the branch, ΦP_n or the allowable strength of the branch, P_n/Ω shall be determined from the limit state of local yielding due to uneven load distribution,

$$\Phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$$

For the overlapping branch and for overlap $25\% \leq O_v \leq 50\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [(O_v/50)(2H_{bi} - 4t_{bi}) + b_{eoi} + b_{eov}] \quad (10.11.2.24)$$

For the overlapping branch, and for overlap $50\% \leq O_v < 80\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + b_{eoi} + b_{eov}] \quad (10.11.2.25)$$

For the overlapping branch and for overlap $80\% \leq O_v \leq 100\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + B_{bi} + b_{eov}] \quad (10.11.2.26)$$

where

b_{eoi} is the *effective width* of the *branch face* welded to the chord,

$$b_{eoi} = [10/(B/t)] [(F_y t)/(F_{ybi} t_{bi})] B_{bi} \leq B_{bi} \quad (10.11.2.27)$$

b_{eov} is the *effective width* of the *branch face* welded to the overlapped brace,

$$b_{eov} = [10/(B_{bj}/t_{bj})] [(F_{ybj} t_{bj})/(F_{ybi} t_{bi})] B_{bi} \leq B_{bi} \quad (10.11.2.28)$$

B_{bi} = overall branch width of the overlapping branch, mm.

B_{bj} = overall branch width of the overlapped branch, mm.

F_{ybi} = *specified minimum yield stress* of the overlapping branch material, MPa.

F_{ybj} = *specified minimum yield stress* of the overlapped branch material, MPa.

H_{bi} = overall depth of the overlapping branch, mm.

t_{bi} = thickness of the overlapping branch, mm.

t_{bj} = thickness of the overlapped branch, mm.

For the overlapped branch, P_n shall not exceed P_n of the overlapping branch, calculated using Equation 10.11.2.24, 10.11.2.25, or 10.11.2.26, as applicable, multiplied by the factor $(A_{bj} F_{ybj} / A_{bi} F_{ybi})$,

where

A_{bi} = cross-sectional area of the overlapping branch

A_{bj} = cross-sectional area of the overlapped branch

10.11.2.3.5 Welds to Branches

The nonuniformity of load transfer along the line of weld, due to differences in relative flexibility of HSS walls in HSS-to-HSS connections, shall be considered in proportioning such welds. This can be considered by limiting the total effective weld length, L_e , of *groove* and *fillet welds* to rectangular HSS as follows:

In T-, Y- and cross-connections,

for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + (B_b - 1.2t_b) \quad (10.11.2.29)$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} \quad (10.11.2.30)$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

In gapped K-connections, around each branch,

for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + 2(B_b - 1.2t_b) \quad (10.11.2.31)$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + (B_b - 1.2t_b) \quad (10.11.2.32)$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

In lieu of the above criteria in Equations 10.11.2.29 to 10.11.2.32, other rational criteria are permitted.

10.11.3 HSS-To-HSS Moment Connections

HSS-to-HSS moment connections are defined as *connections* that consist of one or two *branch members* that are directly welded to a continuous chord that passes through the connection, with the branch or branches loaded by bending moments. A connection shall be classified

As a *T-connection* when there is one branch and it is perpendicular to the chord and as a *Y-connection* when there is one branch but not perpendicular to the chord.

As a *cross-connection* when there is a branch on each (opposite) side of the chord.

For the purposes of this Specification, the centerlines of the branch member(s) and the *chord member* shall lie in a common plane.

10.11.3.1 Definitions of Parameters

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, mm.

B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, mm.

D = outside diameter of round HSS main member, mm.

D_b = outside diameter of round HSS branch member, mm.

F_y = *specified minimum yield stress* of HSS main member, MPa.

F_{yb} = *specified minimum yield stress* of HSS branch member, MPa.

F_u = *ultimate strength* of HSS member, MPa.

H = overall height of rectangular HSS main member, measured in the plane of the connection, mm.

H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, mm.

t = *design wall thickness* of HSS main member, mm.

t_b = *design wall thickness* of HSS branch member, mm.

β = the width ratio; the ratio of branch diameter to chord diameter = D_b / D for round HSS; the ratio of overall branch width to chord width = B_b / B for rectangular HSS

γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D / (2t)$ for round HSS; the ratio of one-half the width to wall thickness = $B / (2t)$ for rectangular HSS

η = the *load length parameter*, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N / B , where $N = H_b / \sin \theta$

θ = acute angle between the branch and chord (degrees)

10.11.3.2 Criteria for Round HSS

The interaction of *stress* due to *chord member forces* and local branch *connection forces* shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \quad (10.11.3.1)$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \quad (10.11.3.2)$$

and

P_r = required axial strength in chord, N.

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, N

$M_r = M_u$ = required flexural strength in chord, using *LRFD load combinations*, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, N

$M_r = M_a$ = required flexural strength in chord, using *ASD load combinations*, N-mm.

$F_c = 0.6 F_y$, MPa.

10.11.3.2.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

Branch angle: $\theta \geq 30^\circ$

Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for *T*- and *Y*-connections; less than or equal to 40 for *cross-connections*

Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50

Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05 E/F_y$

Width ratio: $0.2 < D_b/D \leq 1.0$

Strength: $F_y \leq 360$ MPa. for chord and branches

Ductility: $F_u/F_y \leq 0.8$

10.11.3.2.2 Branches with In-Plane Bending Moments in T-, Y- and Cross-Connections

The *design strength*, ϕM_n and the *allowable strength*, M_n/Ω , shall be the lowest value obtained according to the *limit states of chord plastification and shear yielding (punching)*.

For the limit state of chord plastification,

$$M_n \sin \theta = 5.39 F_y t^2 \gamma^{0.5} \beta D_b Q_f \quad (10.11.3.3)$$

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(1 + 3 \sin \theta)/4 \sin^2 \theta] \quad (10.11.3.4)$$

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

10.11.3.2.3 Branches with Out-of-Plane Bending Moments in T-, Y- and Cross-Connections

The *design strength*, ΦM_n and the *allowable strength*, M_n/Ω , shall be the lowest value obtained according to the *limit states of chord plastification and shear yielding (punching)*.

For the limit state of chord plastification,

$$M_n \sin \theta = F_y t^2 D_b [3.0/(1 - 0.81\beta)] Q_f \quad (10.11.3.5)$$

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For the limit state of shear yielding (punching),

$$M_n = 0.6 F_y t D_b^2 [(3 + \sin \theta)/4 \sin^2 \theta] Q_f \quad (10.11.3.6)$$

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

10.11.3.2.4 Branches with Combined Bending Moment and Axial Force in T-, Y- and Cross-Connections

Connections subject to branch axial load, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these *load effects*, should satisfy the following.

For design according to Section 10.2.3.3 (LRFD):

$$(P_r/\phi P_n) + (M_{r-ip}/\phi M_{n-ip})^2 + (M_{r-op}/\phi M_{n-op}) \leq 1.0 \quad (10.11.3.7)$$

where

$P_r = P_u$ = required axial strength in branch, using LRFD load combinations, N

ϕP_n = design strength obtained from Section 10.11.2.2.2

M_{r-ip} = required in-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-ip} = design strength obtained from Section 10.11.3.2.2

M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-op} = design strength obtained from Section 10.11.3.2.3

For design according to Section 10.2.3.4 (ASD):

$$(P_r/(P_n/\Omega)) + (M_{r-ip}/(M_{n-ip}/\Omega))^2 + (M_{r-op}/(M_{n-op}/\Omega)) \leq 1.0 \quad (10.11.3.8)$$

where

$P_r = P_a$ = required axial strength in branch, using ASD load combinations, N

P_n/Ω = allowable strength obtained from Section 10.11.2.2.2

M_{r-ip} = required in-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-ip}/Ω = allowable strength obtained from Section 10.11.3.2.2

M_{r-op} = required out-of-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-op}/Ω = allowable strength obtained from Section 10.11.3.2.3

10.11.3.3 Criteria for Rectangular HSS

The interaction of *stress due to chord member forces* and local branch *connection forces* shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = (1.3 - 0.4U/\beta) \leq 1 \quad (10.11.3.9)$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \quad (10.11.3.10)$$

and

P_r = required axial strength in chord, N.

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm²

F_c = available stress, MPa.

S = chord elastic section modulus, mm³.

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using LRFD load combinations, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using ASD load combinations, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6 F_y$, MPa.

10.11.3.3.1 Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits:

Branch angle is approximately 90°

Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35

Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35

Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35

Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25

Aspect ratio: $0.5 \leq \text{ratio of depth to width} \leq 2.0$

Strength: $F_y \leq 360$ MPa. for chord and branches

Ductility: $F_y / F_u \leq 0.8$

Other limits apply for specific criteria

10.11.3.3.2 Branches with In-Plane Bending Moments in T- and Cross-Connections

The *design strength*, ϕM_n , and the *allowable strength*, M_n/Ω , shall be the lowest value obtained according to the *limit states of chord wall plastification*, *sidewall local yielding* and local yielding due to *uneven load distribution*.

For the limit state of chord wall plastification,

$$M_n = F_y t^2 H_b [(1/2\eta) + 2/(1 - \beta)^{0.5} + \eta/(1 - \beta)] Q_f \quad (10.11.3.11)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

This limit state need not be checked when $\beta > 0.85$.

For the limit state of sidewall local yielding,

$$M_n = 0.5 F_y^* t (H_b + 5t)^2 \quad (10.11.3.12)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

where

$F_y^* = F_y$ for *T-connections*

$F_y^* = 0.8 F_y$ for *cross-connections*

This limit state need not be checked when $\beta < 0.85$.

For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb} [Z_b - (1 - b_{eoi}/B_b) B_b H_b t_b] \quad (10.11.3.13)$$

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

Where

$$b_{eoi} = [10/(B/t)] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (10.11.3.14)$$

Z_b = branch plastic section modulus about the axis of bending, mm^3 .

This limit state need not be checked when $\beta < 0.85$.

10.11.3.3.3 Branches with Out-of-Plane Bending Moments in T- and Cross-Connections

The *design strength*, ϕM_n , and the *allowable strength*, M_n/Ω , shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding, local yielding due to uneven load distribution and chord *distortional failure*.

For the limit state of chord wall plastification,

$$M_n = F_y t^2 [0.5 H_b (1 + \beta)/(1 - \beta) + [2 B B_b (1 + \beta)/(1 - \beta)]^{0.5}] Q_f \quad (10.11.3.15)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

This limit state need not be checked when $\beta > 0.85$.

For the limit state of sidewall local yielding,

$$M_n = F_y^* t (B - t) (H_b + 5t) \quad (10.11.3.16)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

where

$F_y^* = F_y$ for T-connections

$F_y^* = 0.8 F_y$ for cross-connections

This limit state need not be checked when $\beta < 0.85$.

For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb} [Z_b - 0.5(1 - b_{eoi}/B_b)^2 B_b^2 t_b] \quad (10.11.3.17)$$

$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

where

$$b_{eoi} = [10/(B/t)] [F_y t / (F_{yb} t_b)] B_b \leq B_b \quad (10.11.3.18)$$

Z_b = branch plastic section modulus about the axis of bending, mm³.

This limit state need not be checked when $\beta < 0.85$.

For the limit state of chord distortional failure,

$$M_n = 2F_y t [H_b t + [BHt(B + H)]^{0.5}] \quad (10.11.3.19)$$

$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

This limit state need not be checked for cross-connections or for T-connections if chord distortional failure is prevented by other means.

10.11.3.3.4 Branches with Combined Bending Moment and Axial Force in T- and Cross-Connections

Connections subject to branch axial *load*, branch in-plane bending moment and branch out-of-plane bending moment, or any combination of these *load effects*, should satisfy

For design according to Section 10.2.3.3 (LRFD)

$$(P_r / \phi P_n) + (M_{r-ip} / \phi M_{n-ip}) + (M_{r-op} / \phi M_{n-op}) \leq 1.0 \quad (10.11.3.20)$$

where

$P_r = P_u$ = required axial strength in branch, using LRFD load combinations, N

ϕP_n = design strength obtained from Section 10.11.2.3.2

M_{r-ip} = required in-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-ip} = design strength obtained from Section 10.11.3.3.2

M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm.

ϕM_{n-op} = design strength obtained from Section 10.11.3.3.3

For design according to Section 10.2.3.4 (ASD)

$$(P_r / (P_n / \Omega)) + (M_{r-ip} / (M_{n-ip} / \Omega)) + (M_{r-op} / (M_{n-op} / \Omega)) \leq 1.0 \quad (10.11.3.21)$$

where

$P_r = P_a$ = required axial strength in branch, using ASD load combinations, N

P_n / Ω = allowable strength obtained from Section 10.11.2.3.2

M_{r-ip} = required in-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-ip} / Ω = allowable strength obtained from Section 10.11.3.3.2

M_{r-op} = required out-of-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-op} / Ω = allowable strength obtained from Section 10.11.3.3.3

10.12 Design for Serviceability

This chapter addresses *serviceability* performance design requirements.

10.12.1 General Provisions

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Limiting values of structural behavior for serviceability (for example, maximum deflections, accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using appropriate *load combinations* for the serviceability *limit states* identified.

10.12.2 Camber

Where *camber* is used to achieve proper position and location of the structure, the magnitude, direction and location of camber shall be specified in the structural drawings in accordance with the provisions of Chapter 1.

10.12.3 Deflections

Deflections in structural members and structural systems under appropriate *service load combinations* shall not impair the *serviceability* of the structure. Limiting values of deflections of various structural members shall be in accordance with those specified in Section 1.4 of Chapter 1.

10.12.4 Drift

Drift of a structure shall be evaluated under *service loads* to provide for *serviceability* of the structure, including the integrity of interior partitions and exterior *cladding*. *Drift* under strength *load combinations* shall not cause collision with adjacent structures or exceed the limiting values specified in Section 1.5.6 of Chapter 1.

10.12.5 Vibration

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. Sources of vibration to be considered include pedestrian loading, vibrating machinery and others identified for the structure. It must be shown by any rational method of analysis that the vibrations induced by any source including the above mentioned ones is within tolerable limit and shall not cause any adverse effect on the safety, stability and durability of the structure.

10.12.6 Wind-Induced Motion

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered. For flexible building and structures as defined in Sec. 2.4.2, it must be shown by a rational dynamic analysis that wind induced vibration does not cause any discomfort to occupants as well as the wind induced dynamic effect does not cause any adverse effect on the safety, stability and durability of the structure.

10.12.7 Expansion and Contraction

The effects of thermal expansion and contraction of a building shall be considered. Damage to building *cladding* can cause water penetration and may lead to corrosion.

10.12.8 Connection Slip

The effects of *connection* slip shall be included in the design where slip at bolted connections may cause deformations that impair the *serviceability* of the structure. Where appropriate, the connection shall be designed to preclude slip. For the design of slip-critical connections see Sections 10.10.3.8 and 10.10.3.9.

10.13 Fabrication, Erection and Quality Control

This chapter addresses requirements for design and shop drawings, fabrication, shop painting, erection and quality control.

10.13.1 DESIGN DRAWINGS AND SPECIFICATIONS

Structural Design Drawings and Specifications

Unless otherwise indicated in the contract documents, the structural design drawings shall be based upon consideration of the design loads and forces to be resisted by the structural steel frame in the completed project.

The structural design drawings shall clearly show the work that is to be performed and shall give the following information with sufficient dimensions to accurately convey the quantity and nature of the structural steel to be fabricated:

- a) The size, section, material grade and location of all members;
- b) All geometry and working points necessary for layout;
- c) Floor elevations;
- d) Column centers and offsets;
- e) The camber requirements for members;
- f) Joining requirements between elements of built-up members; and,
- g) The information that is required in Sections 10.13.1.1.1 through 10.13.1.1.6.

The structural steel specifications shall include any special requirements for the fabrication and erection of the structural steel.

The structural design drawings, specifications and addenda shall be numbered and dated for the purposes of identification.

10.13.1.1 Detailing of Components

Permanent bracing, column stiffeners, column web doubler plates, bearing stiffeners in beams and girders, web reinforcement, openings for other trades and other special details, where required, shall be shown in sufficient detail in the structural design drawings so that the quantity, detailing and fabrication requirements for these items can be readily understood.

10.13.1.2 Designer's Responsibility

The owner's designated representative for design shall indicate one of the following options for each connection:

- (1) The complete connection design shall be shown in the structural design drawings;
- (2) In the structural design drawings or specifications, the connection shall be designated to be selected or completed by an experienced steel detailer; or,
- (3) In the structural design drawings or specifications, the connection shall be designated to be designed by a licensed professional engineer working for the fabricator.

In all of the above options,

- (a) The requirements of Section 10.13.1.1 shall apply; and,
- (b) The approvals process in Section 10.13.2.4 shall be followed.

When option (2) above is specified:

The experienced steel detailer shall utilize tables or schematic information provided in the structural design drawings in the selection or completion of the connections. When such information is not provided, standard reference information as approved by the owner's designated representative for design, shall be used.

When option (2) or (3) above is specified

The owner's designated representative for design shall provide the following information in the structural design drawings and specifications:

- a) Any restrictions on the types of connections that are permitted;
- b) Data concerning the loads, including shears, moments, axial forces and transfer forces, that are to be resisted by the individual members and their connections, sufficient to allow the selection, completion, or design of the connection details while preparing the shop and erection drawings;
- c) Whether the data required in (b) is given at the service-load level or the factored-load level;
- d) Whether LRFD or ASD is to be used in the selection, completion, or design of connection details; and,
- e) What substantiating connection information, if any, is to be provided with the shop and erection drawings to the owner's designated representative for design.

When option (3) above is specified:

- a) The fabricator shall submit in a timely manner representative samples of the required substantiating connection information to the owner's designated representatives for design and construction. The owner's designated representative for design shall confirm in writing in a timely manner that these representative samples are consistent with the requirements in the contract documents, or shall advise what modifications are required to bring the representative samples into compliance with the requirements in the contract documents. This initial submittal and review is in addition to the requirements in Section 10.13.2.4.
- b) The licensed professional engineer in responsible charge of the connection design shall review and confirm in writing as part of the substantiating connection information, that the shop and erection drawings properly incorporate the connection designs. However, this review by the licensed professional engineer in responsible charge of the connection design does not replace the approval process of the shop and erection drawings by the owner's designated representative for design in Section 10.13.2.4.
- c) The fabricator shall provide a means by which the substantiating connection information is referenced to the related connections on the shop and erection drawings for the purpose of review.

10.13.1.2.1 Levelling Plates

When leveling plates are to be furnished as part of the contract requirements, their locations and required thickness and sizes shall be specified in the contract documents.

10.13.1.2.2 Non-Structural Elements

When the structural steel frame, in the completely erected and fully connected state, requires interaction with non-structural steel elements (see Section 2) for strength and/or stability, those non-structural steel elements shall be identified in the contract documents as required in Section 10.13.5.10.

10.13.1.2.3 Camber

When camber is required, the magnitude, direction and location of camber shall be specified in the structural design drawings.

10.13.1.2.4 Painting Information

Specific members or portions thereof that are to be left unpainted shall be identified in the contract documents. When shop painting is required, the painting requirements shall be specified in the contract documents, including the following information:

- a) The identification of specific members or portions thereof to be painted;
- b) The surface preparation that is required for these members;
- c) The paint specifications and manufacturer's product identification that are required for these members; and,
- d) The minimum dry-film shop-coat thickness that is required for these members.

10.13.1.3 Architectural, Electrical and Mechanical Design Drawings and Specifications

All requirements for the quantities, sizes and locations of structural steel shall be shown or noted in the structural design drawings. The use of architectural, electrical and/or mechanical design drawings as a supplement to the structural design drawings is permitted for the purposes of defining detail configurations and construction information.

10.13.1.4 Discrepancies

When discrepancies exist between the design drawings and specifications, the design drawings shall govern. When discrepancies exist between scale dimensions in the design drawings and the figures written in them, the figures shall govern. When discrepancies exist between the structural design drawings and the architectural, electrical or mechanical design drawings or design drawings for other trades, the structural design drawings shall govern.

When a discrepancy is discovered in the contract documents in the course of the fabricator's work, the fabricator shall promptly notify the owner's designated representative for construction so that the discrepancy can be resolved by the owner's designated representative for design. Such resolution shall be timely so as not to delay the fabricator's work. See Sections 10.13.1.5 and 10.13.7.3.

10.13.1.5 Legibility of Design Drawings

Design drawings shall be clearly legible and drawn to an identified scale that is appropriate to clearly convey the information.

10.13.1.6 Revisions to the Design Drawings and Specifications

Revisions to the design drawings and specifications shall be made either by issuing new design drawings and specifications or by reissuing the existing design drawings and specifications. In either case, all revisions, including revisions that are communicated through responses to RFIs or the annotation of shop and/or erection drawings (see Section 10.13.2.4.2), shall be clearly and individually indicated in the contract documents. The contract documents shall be dated and identified by revision number. Each design drawings shall be identified by the same drawing number throughout the duration of the project, regardless of the revision. See also Section 10.13.7.3.

10.13.2 Shop and Erection Drawings

Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted *connections*. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

10.13.2.1 Owner Responsibility

The owner shall furnish, in a timely manner and in accordance with the contract documents, complete structural design drawings and specifications that have been released for construction. Unless otherwise noted, design drawings that are provided as part of a contract bid package shall constitute authorization by the owner that the design drawings are released for construction.

Records of the meetings should be written and distributed to all parties. Subsequent meetings to discuss progress and issues that arise during construction also can be helpful, particularly when they are held on a regular schedule.

10.13.2.2 Fabricator Responsibility

Except as provided in Section 10.13.2.5, the fabricator shall produce shop and erection drawings for the fabrication and erection of the structural steel and is responsible for the following:

- (a) The transfer of information from the contract documents into accurate and complete shop and erection drawings; and,
- (b) The development of accurate, detailed dimensional information to provide for the fit-up of parts in the field.

Each shop and erection drawing shall be identified by the same drawing number throughout the duration of the project and shall be identified by revision number and date, with each specific revision clearly identified.

When the fabricator submits a request to change connection details that are described in the contract documents, the fabricator shall notify the owner's designated representatives for design and construction in writing in advance of the submission of the shop and erection drawings. The owner's designated representative for design shall review and approve or reject the request in a timely manner.

When requested to do so by the owner's designated representative for design, the fabricator shall provide to the owner's designated representatives for design and construction its schedule for the submittal of shop and erection drawings so as to facilitate the timely flow of information between all parties.

10.13.2.3 Use of CAD Files and/or Copies of Design Drawings

The fabricator shall neither use nor reproduce any part of the design drawings as part of the shop or erection drawings without the written permission of the owner's designated representative for design. When CAD files or copies of the design drawings are made available for the fabricator's use, the fabricator shall accept this information under the following conditions:

- a) All information contained in the CAD files or copies of the design drawings shall be considered instruments of service of the owner's designated representative for design and shall not be used for other projects, additions to the project or the completion of the project by others. CAD files and copies of the design drawings shall remain the property of the owner's designated representative for design and in no case shall the transfer of these CAD files or copies of the design drawings be considered a sale.
- b) The CAD files or copies of the design drawings shall not be considered to be contract documents. In the event of a conflict between the design drawings and the CAD files or copies thereof, the design drawings shall govern;
- c) The use of CAD files or copies of the design drawings shall not in any way obviate the fabricator's responsibility for proper checking and coordination of dimensions, details, member sizes and fit-up and quantities of materials as required to facilitate the preparation of shop and erection drawings that are complete and accurate as required in Section 4.2; and,
- d) The fabricator shall remove information that is not required for the fabrication or erection of the structural steel from the CAD files or copies of the design drawings.

10.13.2.4 Approval

Except as provided in Section 10.13.2.5, the shop and erection drawings shall be submitted to the owner's designated representatives for design and construction for review and approval. The shop and erection drawings shall be returned to the fabricator within 14 calendar days.

Final substantiating connection information, if any, shall also be submitted with the shop and erection drawings. The owner's designated representative for design is the final authority in the event of a disagreement between parties regarding connection design.

Approved shop and erection drawings shall be individually annotated by the owner's designated representatives for design and construction as either approved or approved subject to corrections noted. When so required, the fabricator shall subsequently make the corrections noted and furnish corrected shop and erection drawings to the owner's designated representatives for design and construction.

10.13.2.4.1 Constituents of Approval

Approval of the shop and erection drawings, approval subject to corrections noted and similar approvals shall constitute the following:

- a) Confirmation that the fabricator has correctly interpreted the contract documents in the preparation of those submittals;
- b) Confirmation that the owner's designated representative for design has reviewed and approved the connection details shown on the shop and erection drawings and submitted in accordance with Sections 10.13.1 and 10.13.2, if applicable; and,
- c) Release by the owner's designated representatives for design and construction for the fabricator to begin fabrication using the approved submittals.

Such approval shall not relieve the fabricator of the responsibility for either the accuracy of the detailed dimensions in the shop and erection drawings or the general fit-up of parts that are to be assembled in the field.

The fabricator shall determine the fabrication schedule that is necessary to meet the requirements of the contract.

10.13.2.4.2 Authorization by Owner

Unless otherwise noted, any additions, deletions or revisions that are indicated in responses to RFIs or on the approved shop and erection drawings shall constitute authorization by the owner that the additions, deletions or revisions are released for construction. The fabricator and the erector shall promptly notify the owner's designated representative for construction when any direction or notation in responses to RFIs or on the shop

or erection drawings or other information will result in an additional cost and/or a delay. See Sections 10.13.1.5 and 10.13.7.3.

10.13.2.5 Shop and/or Erection Drawings Not Furnished by the Fabricator

When the shop and erection drawings are not prepared by the fabricator, but are furnished by others, they shall be delivered to the fabricator in a timely manner. These shop and erection drawings shall be prepared, insofar as is practical, in accordance with the shop fabrication and detailing standards of the fabricator. The fabricator shall neither be responsible for the completeness or accuracy of shop and erection drawings so furnished, nor for the general fit-up of the members that are fabricated from them.

10.13.2.6 The RFI Process

When requests for information (RFIs) are issued, the process shall include the maintenance of a written record of inquiries and responses related to interpretation and implementation of the contract documents, including the clarifications and/or revisions to the contract documents that result, if any. RFIs shall not be used for the incremental release for construction of design drawings. When RFIs involve discrepancies or revisions, see Sections 10.13.1.3, 10.13.1.5, and 10.13.2.4.2.

10.13.2.7 Erection Drawings

Erection drawings shall be provided to the erector in a timely manner so as to allow the erector to properly plan and perform the work.

10.13.3 MATERIALS

10.13.3.1 Mill Materials

Unless otherwise noted in the contract documents, the fabricator is permitted to order the materials that are necessary for fabrication when the fabricator receives contract documents that have been released for construction.

Unless otherwise specified by means of special testing requirements in the contract documents, mill testing shall be limited to those tests that are required for the material in the ASTM specifications indicated in the contract documents. Materials ordered to special material requirements shall be marked by the supplier as specified in ASTM A6/A6M Section 12 prior to delivery to the fabricator's shop or other point of use. Such material not so marked by the supplier, shall not be used until:

- a) Its identification is established by means of testing in accordance with the applicable ASTM specifications; and,
 - b) A fabricator's identification mark, as described in Section 10.13.2 has been applied.
- When mill material does not satisfy ASTM A6/A6M tolerances for camber, profile, flatness or sweep, the fabricator shall be permitted to perform corrective procedures, including the use of controlled heating and/or mechanical straightening, subject to the limitations in the AISC Specification.

10.13.3.2 Stock Materials

If used for structural purposes, materials that are taken from stock by the fabricator shall be of a quality that is at least equal to that required in the ASTM specifications indicated in the contract documents.

Material test reports shall be accepted as sufficient record of the quality of materials taken from stock by the fabricator. The fabricator shall review and retain the material test reports that cover such stock materials. However, the fabricator need not maintain records that identify individual pieces of stock material against individual material test reports, provided the fabricator purchases stock materials that meet the requirements for material grade and quality in the applicable ASTM specifications.

Stock materials that are purchased under no particular specification, under a specification that is less rigorous than the applicable ASTM specifications or without material test reports or other recognized test reports shall not be used without the approval of the owner's designated representative for design.

10.13.4 Fabrication

10.13.4.1 Cambering, Curving and Straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct *camber*, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 593⁰ C for A514/A514M and A852/A852M steel nor 1,200⁰ F (649⁰ C) for other steels.

10.13.4.2 Thermal Cutting

Thermally cut edges shall meet the requirements of AWS D1.1, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will be subject to calculated static tensile *stress* shall be free of round-bottom *gouges* greater than 5 mm deep and sharp V-shaped notches. *Gouges* deeper than 5 mm and notches shall be removed by grinding or repaired by welding.

Reentrant corners, except reentrant corners of *beam copes* and weld access holes, shall meet the requirements of AWS D1.1, Section A5.16. If another specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Section 10.10.1.6. Beam copes and weld access holes in shapes that are to be galvanized shall be ground. For shapes with a flange thickness not exceeding 50 mm the roughness of *thermally cut* surfaces of copes shall be no greater than a surface roughness value of 50 μ m as defined in ASME B46.1 Surface Texture (*Surface Roughness, Waviness, and Lay*). For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm and welded built-up shapes with material thickness greater than 50 mm, a preheat temperature of not less than 66⁰ C shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm and built-up shapes with a material thickness greater than 50 mm shall be ground and inspected for cracks using magnetic particle inspection in accordance with ASTM E709. Any crack is unacceptable regardless of size or location.

10.13.4.3 Planing of Edges

Planing or finishing of sheared or *thermally cut* edges of plates or shapes is not required unless specifically called for in the contract documents or included in a stipulated edge preparation for welding.

10.13.4.4 Welded Construction

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section J2.

10.13.4.5 Bolted Construction

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a *drift* pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 10.13.3.3 except that *thermally cut* holes shall be permitted with a surface roughness profile not exceeding 25 μ m as defined in ASME B46.1. *Gouges* shall not exceed a depth of 2 mm.

Fully inserted finger *shims*, with a total thickness of not more than 6 mm within a *joint* are permitted in *joints* without changing the strength (based upon hole type) for the design of *connections*. The orientation of such *shims* is independent of the direction of application of the *load*.

The use of high-strength bolts shall conform to the requirements of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, except as modified in Section 10.10.3.

10.13.4.6 Compression Joints

Compression *joints* that depend on contact bearing as part of the *splice* strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means.

10.13.4.7 Dimensional Tolerances

Dimensional tolerances shall be in accordance with ASTM A6/A6M.

10.13.4.8 Finish of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

(1) Steel bearing plates 50 mm or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 50 mm but not over 100 mm in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 100 mm in thickness shall be milled for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and *column* bases that are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when complete-*joint*- penetration *groove welds* are provided between the *column* and the bearing plate.

10.13.4.9 Holes for Anchor Rods

Holes for anchor rods shall be permitted to be *thermally cut* in accordance with the provisions of Section 10.13.2.2.

10.13.4.10 Drain Holes

When water can collect inside *HSS* or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base, or protected by other suitable means.

10.13.4.11 Requirements for Galvanized Members

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure build-up in enclosed parts.

10.13.5 Shop Painting

10.13.5.1 General Requirements

Shop paint is not required unless specified by the contract documents.

10.13.5.2 Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

10.13.5.3 Contact Surfaces

Paint is permitted in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, Section 3.2.2(b).

10.13.5.4 Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

10.13.5.5 Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 50 mm of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

10.13.6 Erection

10.13.6.1 Alignment of Column Bases

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

10.13.6.2 Bracing

The frame of steel skeleton buildings shall be carried up true and plumb. Temporary bracing shall be provided, wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

10.13.6.3 Alignment

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

10.13.6.4 Fit of Column Compression Joints and Base Plates

Lack of contact bearing not exceeding a gap of 2 mm, regardless of the type of *splice* used (*partial-joint-penetration groove welded* or bolted), is permitted. If the gap exceeds 2 mm, but is less than 6 mm, and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel *shims*. Shims need not be other than mild steel, regardless of the grade of the main material.

10.13.6.5 Field Welding

Shop paint on surfaces adjacent to *joints* to be field welded shall be wire brushed if necessary to assure weld quality.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

10.13.6.6 Field Painting

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

10.13.6.7 Connections

As erection progresses, the structure shall be securely bolted or welded to support the dead, wind and erection loads.

10.13.7 Quality Control

The fabricator shall provide *quality control* procedures to the extent that the fabricator deems necessary to assure that the work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

10.13.7.1 Cooperation

As far as possible, the inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

10.13.7.2 Rejections

Material or workmanship not in conformance with the provisions of this Specification may be rejected at any time during the progress of the work.

The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

10.13.7.3 Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section 10.10.2.

When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.

When nondestructive testing is required, the process, extent and standards of acceptance shall be clearly defined in the design documents.

10.13.7.4 Inspection of Slip-Critical High-Strength Bolted Connections

The inspection of slip-critical high-strength bolted *connections* shall be in accordance with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

10.13.7.5 Identification of Steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material identification, visible at least through the “fit-up” operation, for the main structural elements of each shipping piece.

10.14 Direct Analysis Method

This section addresses the *direct analysis method* for structural systems comprised of *moment frames, braced frames, shear walls*, or combinations thereof.

10.14.1 General Requirements

Members shall satisfy the provisions of Section 10.8.1 with the nominal *column* strengths, P_n , determined using $K = 1.0$. The *required strengths* for members, *connections* and other structural elements shall be determined using a second-order *elastic analysis* with the constraints presented in Section 10.14.3. All component and *connection* deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

10.14.2 Notional Loads

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections, inelasticity, or both. *Notional loads* are *lateral loads* that are applied at each framing level and specified in terms of the *gravity loads* applied at that level. The *gravity load* used to determine the *notional load* shall be equal to or greater than the *gravity load* associated with the *load combination* being evaluated. *Notional loads* shall be applied in the direction that adds to the destabilizing effects under the specified *load combination*.

10.14.3 Notional Loads

(1) The *second-order analysis* shall consider both $P-\delta$ and $P-\Delta$ effects. It is permitted to perform the analysis using any general second-order analysis method, or by the amplified *first-order analysis* method of Section 10.3.2, provided that the B_1 and B_2 factors are based on the reduced *stiffnesses* defined in Equations 10.14.3.2 and 10.14.3.3. Analyses shall be conducted according to the design and loading requirements specified in either Section 10.2.3.3 (LRFD) or Section 10.2.3.4 (ASD). For ASD, the second-order analysis shall be carried out under 1.6 times the ASD *load combinations* and the results shall be divided by 1.6 to obtain the *required strengths*.

Methods of analysis that neglect the effects of $P-\delta$ on the lateral displacement of the structure are permitted where the axial *loads* in all members whose flexural stiffnesses are considered to contribute to the lateral *stability* of the structure satisfy the following limit:

$$\alpha P_r < 0.15 P_{eL} \quad (10.14.3.1)$$

where

P_r = required axial compressive strength under LRFD or ASD *load combinations*, N

$P_{eL} = \pi^2 EI / L^2$, evaluated in the plane of bending

And $\alpha = 1.0$ (LRFD) $\alpha = 1.6$ (ASD)

(2) A *notional load*, $N_i = 0.002 Y_i$, applied independently in two orthogonal directions, shall be applied as a *lateral load* in all load combinations. This load shall be in addition to other lateral loads, if any,

where

N_i = notional lateral load applied at level i , N

Y_i = *gravity load* from the LRFD *load combination* or 1.6 times the ASD *load combination* applied at level i , N

The notional load coefficient of 0.002 is based on an assumed initial story out-of-plumbness ratio of 1/500. Where a smaller assumed out-of-plumbness is justified, the notional load coefficient may be adjusted proportionally.

For frames where the ratio of second-order drift to first-order drift is equal to or less than 1.5, it is permissible to apply the notional load, N_i , as a minimum lateral load for the gravity-only load combinations and not in combination with other lateral loads.

For all cases, it is permissible to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying a notional load or a minimum lateral load as defined above.

(3) A reduced flexural stiffness, EI^* ,

$$EI^* = 0.08 \tau_b EI \quad (10.14.3.2)$$

shall be used for all members whose flexural stiffness is considered to contribute to the lateral *stability* of the structure,

where

I = moment of inertia about the axis of bending, mm^4

$\tau_b = 1.0$ for $\alpha P_r / P_y \leq 0.5$

$= 4[\alpha P_r / P_y (1 - \alpha P_r / P_y)]$ for $\alpha P_r / P_y > 0.5$

P_r = required axial compressive strength under LRFD or ASD load combinations, N

$P_y = AF_y$, member yield strength, N

And $\alpha = 1.0$ (LRFD) $\alpha = 1.6$ (ASD)

In lieu of using $\tau_b < 1.0$ where $\alpha P_r / P_y > 0.5$, $\tau_b = 1.0$ may be used for all members, provided that an additive *notional load* of $0.001Y_i$ is added to the notional load required in (2).

(4) A reduced flexural stiffness, EA^* ,

$$EA^* = 0.8 EA \quad (10.14.3.3)$$

shall be used for members whose axial stiffness is considered to contribute to the lateral *stability* of the structure, where A is the cross-sectional member area.

10.15 Inelastic Analysis and Design

10.15.1 General Provisions

Inelastic analysis is permitted for design according to the provisions of Section 10.2.3.3 (LRFD). Inelastic analysis is not permitted for design according to the provisions of Section 10.2.3.4 (ASD) except as provided in Section 10.15.3.

10.15.2 Materials

Members undergoing plastic hinging shall have a *specified minimum yield stress* not exceeding 450 MPa.

10.15.3 Moment Redistribution

Beams and girders composed of *compact sections* as defined in Section 10.2.4 and satisfying the *unbraced length* requirements of Section 10.15.7, including *composite* members, may be proportioned for nine-tenths of the negative moments at points of support, produced by the *gravity loading* computed by an *elastic analysis*, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for moments produced by loading on cantilevers and for design according to Sections 10.15.4 through 10.15.8 of this section.

If the negative moment is resisted by a *column* rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial *force* and flexure, provided that the axial force does not exceed $0.15\phi_c F_y A_g$ for LRFD or $0.15 F_y A_g / \Omega_c$ for ASD,

where

A_g = gross area of member, mm^2

F_y = *specified minimum yield stress* of the compression flange, MPa.

ϕ_c = *resistance factor* for compression = 0.90

Ω_c = *safety factor* for compression = 1.67

10.15.4 Local Buckling

Flanges and webs of members subject to plastic hinging in combined flexure and axial compression shall be compact with width-thickness ratios less than or equal to the limiting λ_p defined in Table 10.2.4.1 or as modified as follows:

- a) For webs of doubly symmetric wide flange members and rectangular HSS in combined flexure and compression

(i) For $P_u/(\phi_b P_y) \leq 0.125$

$$h/t_w \leq 3.76 \sqrt{\frac{E}{F_y}} \left(1 - \frac{2.75 P_u}{\phi_b P_y} \right) \quad (10.15.4.1)$$

(ii) For $P_u/\phi_b P_y > 0.125$

$$h/t_w \leq 1.12 \sqrt{\frac{E}{F_y}} \left(2.33 - \frac{P_u}{\phi_b P_y} \right) \geq 1.49 \sqrt{\frac{E}{F_y}} \quad (10.15.4.2)$$

where

E = modulus of elasticity of steel 200 000 MPa.

F_y = specified minimum yield stress of the type of steel being used, MPa.

h = as defined in Section 10.2.4.2, mm.

P_u = required axial strength in compression, N.

P_y = member yield strength, N.

t_w = web thickness, mm.

ϕ_b = resistance factor for flexure = 0.90

- b) For flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression, flange *cover plates*, and *diaphragm plates* between lines of *fasteners* or welds

$$b/t \leq 0.94 \sqrt{E/F_y} \quad (10.15.4.3)$$

where

b = as defined in Section 10.2.4.2, mm.

t = as defined in Section 10.2.4.2, mm.

(c) For circular hollow sections in flexure

$$D/t \leq 0.045 E/F_y \quad (10.15.4.4)$$

where

D = outside diameter of round HSS member, mm.

10.15.5 Stability and Second-Order Effects

Continuous *beams* not subjected to axial loads and that do not contribute to lateral *stability* of framed structures may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis.

Braced frames and *moment frames* may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis provided that *stability* and *second-order effects* are taken into account.

Structures may be designed on the basis of a second-order *inelastic analysis*. For *beam-columns*, *connections* and connected members, the *required strengths* shall be determined from a second-order inelastic analysis, where equilibrium is satisfied on the deformed geometry, taking into account the change in *stiffness* due to yielding

10.15.5.1 Braced Frames

In *braced frames* designed on the basis of *inelastic analysis*, braces shall be designed to remain elastic under the *design loads*. The required axial strength for *columns* and compression braces shall not exceed $\phi_c (0.85 F_y A_g)$,

where

$$\phi_c = 0.90 \text{ (LRFD)}$$

10.15.5.2 Moment Frames

In *moment frames* designed on the basis of *inelastic analysis*, the required axial strength of columns shall not exceed $\phi_c (0.75 F_y A_g)$

where

$$\phi_c = 0.90 \text{ (LRFD)}$$

10.15.6 Columns and Other Compression Members

In addition to the limits set in Sections 10.15.5.1 and 10.15.5.2, the required axial strength of columns designed on the basis of inelastic analysis shall not exceed the design strength, $\phi_c P_n$, determined according to the provisions of Section 10.5.3.

Design by inelastic analysis is permitted if the column slenderness ratio, L/r , does not exceed $4.71\sqrt{E/F_y}$,

where

L = laterally unbraced length of a member, mm.

r = governing radius of gyration, mm.

10.15.7 Beams and Other Flexural Members

The required moment strength, M_u , of *beams* designed on the basis of *inelastic analysis* shall not exceed the *design strength*, ϕM_n , where

$$M_n = M_p = F_y Z < 1.6 F_y S \quad (10.15.7.1)$$

$$\phi_c = 0.90 \text{ (LRFD)}$$

- a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076 \left(\frac{M_1}{M_2} \right) \right] \left(\frac{E}{F_y} \right) r_y \quad (10.15.7.2)$$

where

M_1 = smaller moment at end of unbraced length of beam, N-mm

M_2 = larger moment at end of unbraced length of beam, N-mm

r_y = radius of gyration about minor axis, mm

(M_1 / M_2) is positive when moments cause *reverse curvature* and negative for *single curvature*.

- b) For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10 \left(\frac{M_1}{M_2} \right) \right] \left(\frac{E}{F_y} \right) r_y \geq 0.10 \left(\frac{E}{F_y} \right) r_y \quad (10.15.7.3)$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

10.15.8 Beams and Other Flexural Members

When inelastic analysis is used for symmetric members subject to bending and axial force, the provisions in Section 10.8.1 apply.

Inelastic analysis is not permitted for members subject to torsion and combined torsion, flexure, shear and/or axial force.

10.15.9 Connections

Connections adjacent to plastic hinging regions of connected members shall be designed with sufficient strength and ductility to sustain the *forces* and deformations imposed under the required *loads*.

10.16 Design for Ponding

This section provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding.

10.16.1 Simplified Design for Ponding

The roof system shall be considered stable for *ponding* and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9 C_s \leq 0.25 \quad (10.16.1.1)$$

$$I_d \geq 3940 S^4 \quad (10.16.1.2)$$

where

$$C_p = \frac{504 L_s L_p^4}{I_p}$$

$$C_s = \frac{504 S L_s^4}{I_s}$$

L_p = column spacing in direction of girder (length of primary members), m.

L_s = column spacing perpendicular to direction of girder (length of secondary members), m.

S = spacing of secondary members, m.

I_p = moment of inertia of primary members, mm⁴.

I_s = moment of inertia of secondary members, mm⁴.

I_d = moment of inertia of the steel deck supported on secondary members, mm⁴ per m.

For trusses and steel joists, the moment of inertia I_s shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

10.16.2 Improved Design for Ponding

The provisions given below are permitted to be used when a more exact determination of framing *stiffness* is needed than that given in Section 10.16.1.

For primary members, the stress index shall be

$$U_p = \left(\frac{0.8F_y - f_o}{f_o} \right)_p \quad (10.16.2.1)$$

For secondary members, the stress index shall be

$$U_s = \left(\frac{0.8F_y - f_o}{f_o} \right)_s \quad (10.16.2.2)$$

where

f_o = stress due to the load combination ($D + R$)

D = nominal dead load

R = nominal load due to rainwater or snow, exclusive of the *ponding* contribution, MPa.

For roof framing consisting of primary and secondary members, the combined stiffness shall be evaluated as follows: enter Figure 10.16.1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the

abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

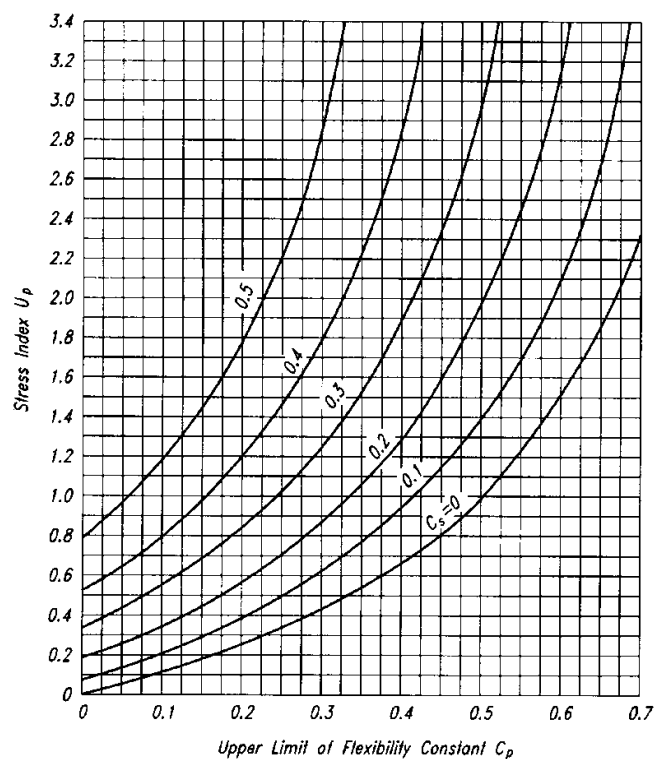


Fig.10.16.1. Limiting flexibility coefficient for the primary systems

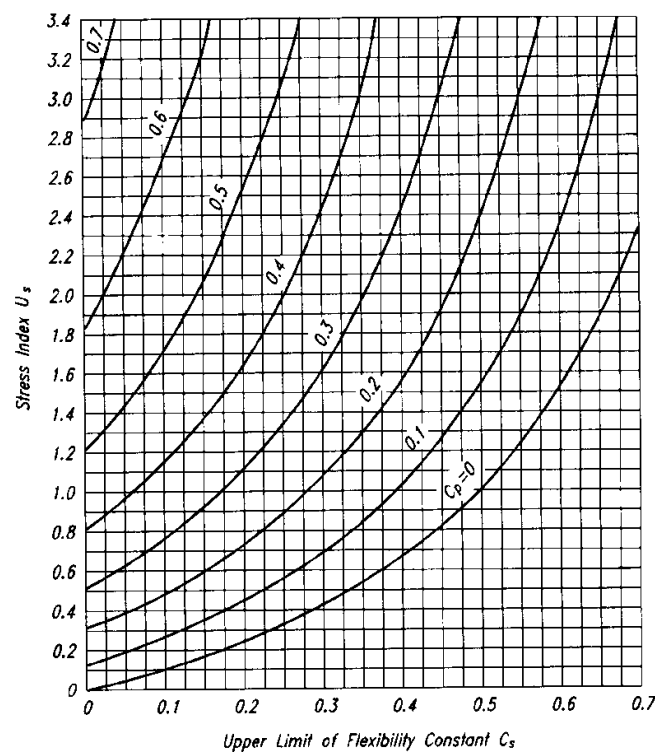


Fig.10.16.2. Limiting flexibility coefficient for the secondary systems.

A similar procedure must be followed using Figure 10.16.2.

For roof framing consisting of a series of equally spaced wall-bearing beams, the stiffness shall be evaluated as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure 10.16.2 with the computed stress index U_s . The limiting value of C_s is determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

For roof framing consisting of metal deck spanning between beams supported on columns, the stiffness shall be evaluated as follows. Employ Figure 10.16.1 or 10.16.2 using as C_s the flexibility constant for a 1 m width of the roof deck ($S = 1.0$).

10.17 Design for Fatigue

This section applies to members and *connections* subject to high cyclic loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the *limit state of fatigue*

10.17.1 General

The provisions of this Section apply to stresses calculated on the basis of service loads. The maximum permitted stress due to unfactored loads is $0.66 F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the service live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration butt welds, the maximum design stress range calculated by Equation 10.17.3.1 applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1.

No evaluation of fatigue resistance is required if the live load stress range is less than the threshold stress range, FTH. See Table 10.17.1.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Section is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this Section is applicable only to structures subject to temperatures not exceeding 150 °C.

The engineer of record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

10.17.2 Calculation of Maximum Stresses and Stress Ranges

Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if any. In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied load.

For members having symmetric cross sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

10.17.3 Design Stress Range

The range of *stress at service loads* shall not exceed the *design stress range* computed as follows.

(a) For stress categories A, B, B', C, D, E and E' (see table 10.17.1) the design stress range, F_{SR} , shall be determined by Equation 10.17.3.1 or 10.17.3.2.

$$F_{SR} = \left(\frac{C_f \times 329}{N} \right)^{0.333} \geq F_{TH} \quad (10.17.3.1)$$

where

F_{SR} = design stress range, MPa.

C_f = constant from Table 10.17.1 for the category

N = number of stress range fluctuations in design life

= number of stress range fluctuations per day $\times$ 365 $\times$ years of design life

F_{TH} = threshold *fatigue stress* range, maximum *stress* range for indefinite design life from Table 10.17.1, MPa.

(b) For stress category F, the design stress range, F_{SR} , shall be determined by Equation 10.17.3.2.

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N} \right)^{0.167} \geq F_{TH} \quad (10.17.3.2)$$

(c) For tension-loaded plate elements connected at their end by cruciform, T, or corner details with *complete-joint-penetration* (CJP) *groove welds* or *partial-joint-penetration* (PJP) *groove welds*, *fillet welds*, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

(i) Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR} , shall be determined by Equation 10.17.3.3 for stress category C which is equal to

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \geq 68.9 \quad (10.17.3.3)$$

(ii) Based upon crack initiation from the root of the weld the design stress range, F_{SR} , on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation 10.17.3.4, stress category C' as follows:

$$F_{SR} = R_{PJP} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \quad (10.17.3.8)$$

where

R_{PJP} is the reduction factor for reinforced or nonreinforced transverse PJP groove welds determined as follows:

$$R_{PJP} = \left(\frac{1.12 - 1.01 \left(\frac{2a}{t_p} \right) + 1.24 \left(\frac{W}{t_p} \right)}{t_p^{0.167}} \right) \leq 1.0 \quad (10.17.3.9)$$

If $R_{PJP} = 1.0$, use stress category C.

$2a$ = the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, mm.

W = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm.

t_p = thickness of tension loaded plate, mm.

(iii) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR} , on the cross section at the toe of the welds shall be determined by Equation 10.17.3.5, stress category C'' as follows:

$$F_{SR} = R_{FIL} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \quad (10.17.3.10)$$

where

R_{FIL} is the reduction factor for joints using a pair of transverse fillet welds only.

$$R_{FIL} = \left(\frac{0.10 + 1.24(w/t_p)}{t_p^{0.167}} \right) \leq 1.0 \quad (10.17.3.11)$$

If $R_{FIL} = 1.0$, use stress category C.

10.17.4 Bolts and Threaded Parts

The range of stress at service loads shall not exceed the stress range computed as follows.

- For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Equation 10.17.3.1 where Cf and FTH are taken from Section 2 of Table 10.17.1.
- For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Equation 10.17.3.1. The factor Cf shall be taken as 3.9×10^8 (as for stress category E'). The threshold stress, FTH shall be taken as 48 MPa (as for stress category D). The net tensile area is given by Equation 10.17.4.1.

$$A_t = \frac{\pi}{4} (d_b - 0.9382P)^2 \quad (10.17.4.1)$$

where

P = pitch, mm per thread

d_b = the nominal diameter (body or shank diameter), mm.

n = threads per mm.

For joints in which the material within the grip is not limited to steel or joints which are not tensioned to the requirements of Table 10.10.3.1, all axial load and moment applied to the joint plus effects of any prying action shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table 10.10.3.1, an analysis of the relative stiffness of the connected parts and bolts shall be permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total service live load and moment plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

10.17.5 Special Fabrication and Erection Requirements

Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long *joints*, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T and corner joints, a reinforcing *fillet weld*, not less than 6 mm in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile *stress* ranges shall not exceed 25 μm , where ASME B46.1 is the reference standard.

Reentrant corners at cuts, *cope*s and weld access holes shall form a radius of not less than 10 mm by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Section 10.10.2.2 for requirements for *end returns* on certain fillet welds subject to cyclic *service loading*.

TABLE 10.17.3.5 – Fatigue Resistance				
Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 m or less, but without reentrant corners.	A	250×10^8	165	Away from all welds or structural connections
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 m or less, but without reentrant corners.	B	120×10^8	110	Away from all welds or structural connections
1.3 Member with drilled or reamed holes. Member with re-entrant corners at <i>cope</i> s, cuts, block-outs or other geometrical discontinuities made to requirements of Section 10.17.3.5, except weld access holes.	B	120×10^8	110	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Section 10.10.1.6 and Section 10.17.3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace force.	C	44×10^8	69	At <i>reentrant</i> corner of weld access hole or at any small hole (may contain bolt for minor connections)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections.	B	120×10^8	110	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120×10^8	110	In net section originating at side of hole

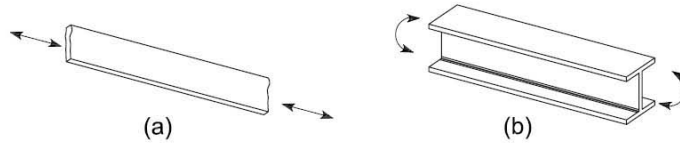
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	22×10^8	48	In net section originating at side of hole
2.4 Base metal at net section of <i>eyebars</i> head or pin plate.	E	11×10^8	31	In net section originating at side of hole

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

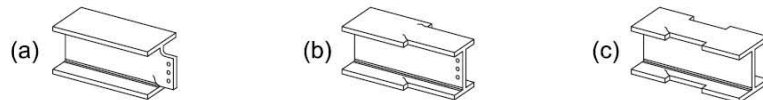
Illustrative Typical Examples

SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING

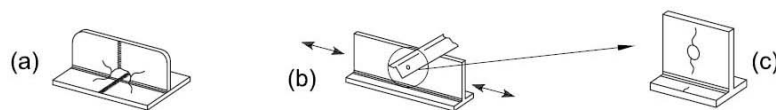
1.1 and 1.2



1.3

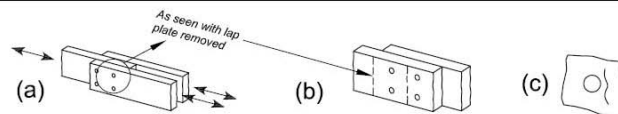


1.4

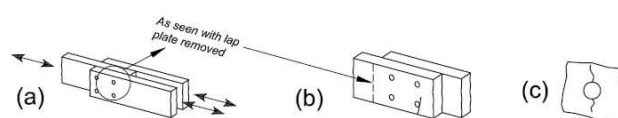


SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

2.1



2.2



2.3



2.4

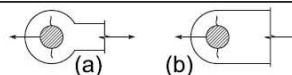


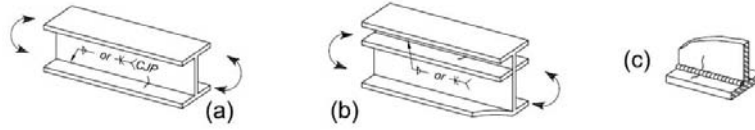
TABLE 10-17.1 (cont.) Fatigue Design Parameters				
Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by	B	120×10^8	110	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete-joint-penetration groove welds with backing bars not removed, or by continuous	B	61×10^8	83	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in	D	22×10^8	48	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	31	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends of coverplates wider than the flange with welds across the ends. Flange thickness ≤ 20 mm Flange thickness > 20 mm				In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide coverplates
3.6 Base metal at ends of partial length welded coverplates wider than the flange without	E'	3.9×10^8	18	In edge of flange at end of coverplate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses. $t \leq 20$ mm $t > 20$ mm	E E'	11×10^8 3.9×10^8	31 18	Initiating from end of any weld termination extending into the base metal

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

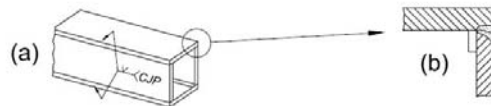
Illustrative Typical Examples

SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS

3.1



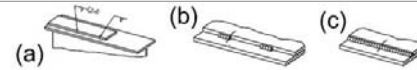
3.2



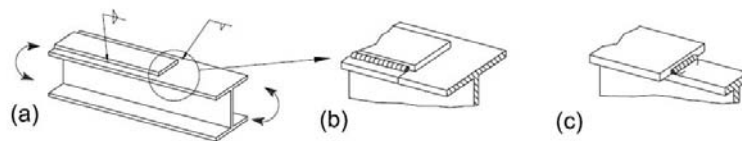
3.3



3.4



3.5



3.6



SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS

4.1

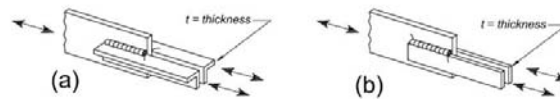


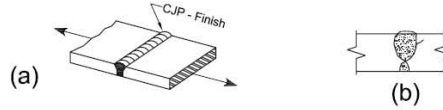
TABLE 10.17.1 (cont.): Fatigue Design Parameters				
Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120 × 108	110	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%. Fy < 620 MPa Fy ≥ 620 MPa	B B'	120 × 108 61 × 108	110 83	From internal discontinuities in filler metal or along fusion boundary or at start of transition when Fy ≥ 620 MPa
5.3 Base metal with Fy equal to or greater than 620 MPa and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 600 mm with the point of tangency at the end of the groove weld.	B	120 × 108	110	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete-joint-penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44 × 108	69	From surface discontinuity at toe of weld extending into base metal or along fusion boundary.
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial-joint-penetration butt or T or corner joints, with reinforcing or contouring fillets, FSR shall be the smaller of the toe crack or root crack stress range. Crack initiating from weld toe: Crack initiating from weld root:	C C'	44 × 108 Eqn. 10.17.3.4	69 None provided	Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

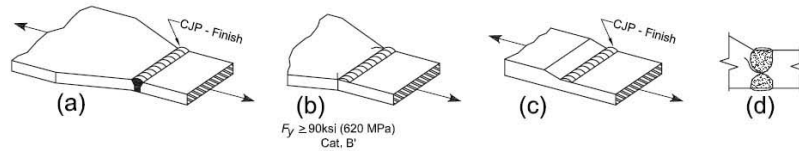
Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

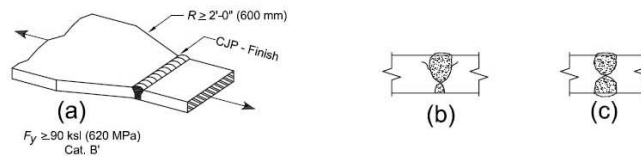
5.1



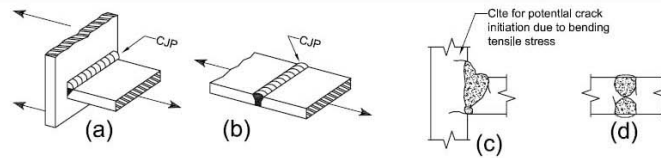
5.2



5.3



5.4



5.5

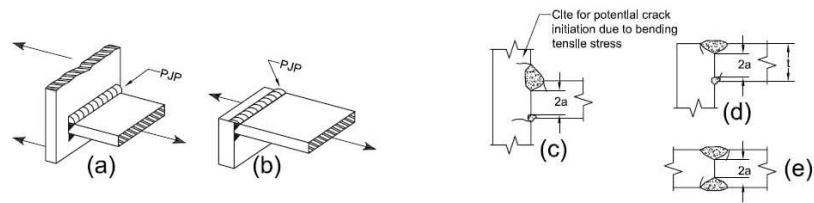


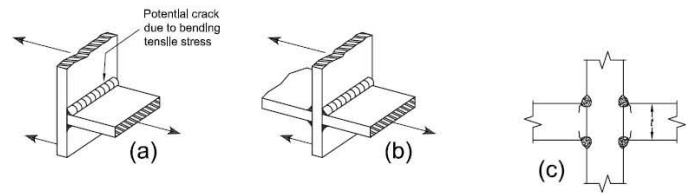
TABLE 10.17.1 (cont.) : Fatigue Design Parameters				
Description	Stress Category	Constant	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range.	C	44×10^8	69	Initiating from geomet- rical discontinuity at toe of weld extend- ing into base metal or, initiating at <i>weld root</i> subject to tension ex- tending up and then out through weld
Crack initiating from weld toe:	C'	Eqn. 10.17.3.5	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44×10^8	69	From geometrical dis- continuity at <i>toe of fil- let</i> extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete-joint- penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq 600$ mm	B	120×10^8	110	
$600 \text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150 \text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50 \text{ mm} > R$	E	11×10^8	31	

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

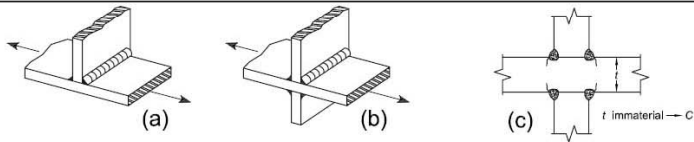
Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



5.7



SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

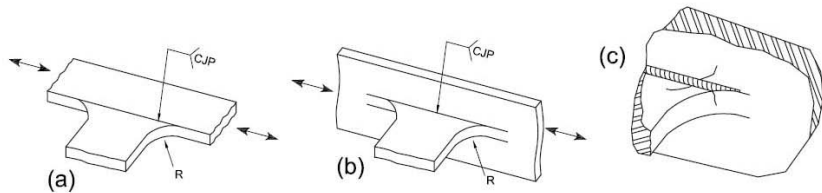


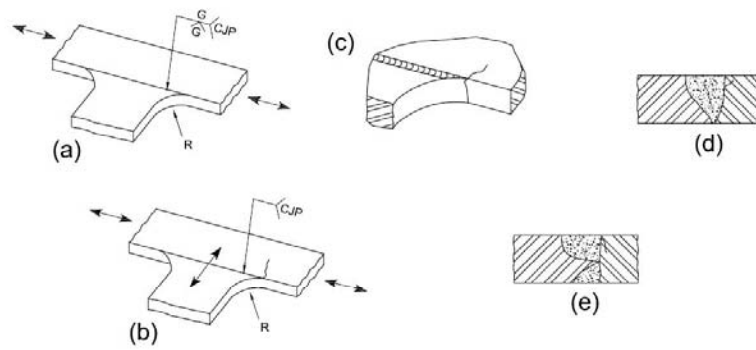
TABLE 10.17.1 (cont.): Fatigue Design Parameters				
Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.2 Base metal at details of equal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth: When weld reinforcement is removed: $R \geq 600$ mm $600 \text{ mm} > R \geq 150$ mm $150 \text{ mm} > R \geq 50$ mm $50 \text{ mm} > R$ When weld reinforcement is not removed: $R \geq 600$ mm $600 \text{ mm} > R \geq 150$ mm $150 \text{ mm} > R \geq 50$ mm $50 \text{ mm} > R$	B C D E C C D E	120×10^8 44×10^8 22×10^8 11×10^8 44×10^8 44×10^8 22×10^8 11×10^8	110 69 48 31 69 69 48 31	Near points of tangency of radius or in the weld or at fusion boundary or member or attachment At toe of the weld either along edge of member or the attachment
6.3 Base metal at details of unequal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth. When weld reinforcement is removed: $R > 50$ mm $R \leq 50$ mm When reinforcement is not removed: Any radius	D E E	22×10^8 11×10^8 11×10^8	48 31 31	At toe of weld along edge of thinner material In weld termination in small radius At toe of weld along edge of thinner material

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.2



6.3

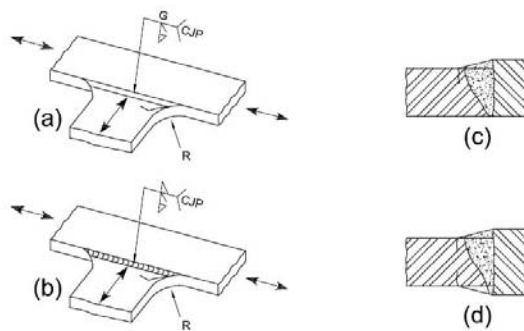


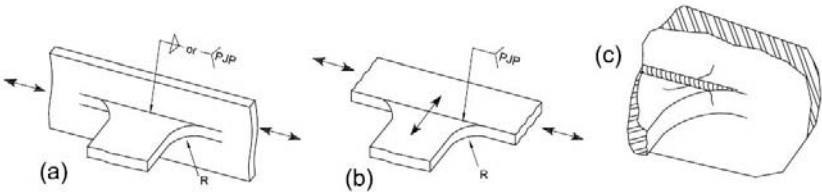
TABLE 6-1. FATIGUE STRESS CATEGORY				
Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, R , with weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 50 \text{ mm}$	D	22×10^8		
$R \leq 50 \text{ mm}$	E	11×10^8		
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS¹				
7.1 Base metal subject to longitudinal loading at details attached by fillet welds parallel or transverse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, a , and attachment height normal to the surface of the member, b :				In the member at the end of the weld
$a < 50 \text{ mm}$	C	44×10^8	69	
$50 \text{ mm} \leq a \leq 12b$ or 100 mm	D	22×10^8	48	
$a > 12b$ or 100 mm when b is $\leq 25 \text{ mm}$	E	11×10^8	31	
$a > 12b$ or 100 mm when b is $> 25 \text{ mm}$	E'	3.9×10^8	18	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial-joint-penetration groove welds, with or without transverse load on detail, when the detail embodies a transition radius, R , with weld termination ground smooth:				In weld termination extending into member
$R > 50 \text{ mm}$	D	22×10^8		
$R \leq 50 \text{ mm}$	E	11×10^8		
¹ "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the stress flow in the member and thus reduces the fatigue resistance.				

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

Illustrative Typical Examples

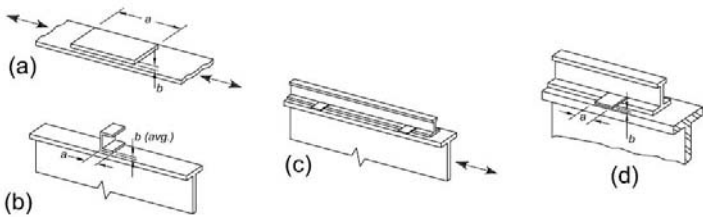
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.4



SECTION 7 – BASE METAL AT SHORT ATTACHMENTS

7.1



7.2

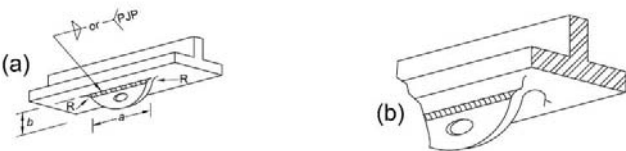
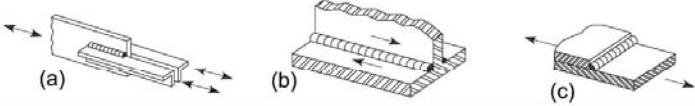
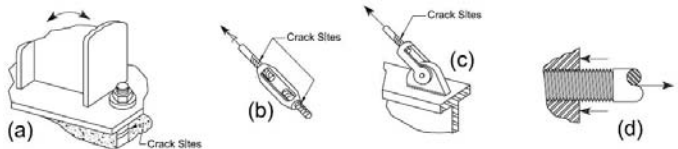


TABLE 10.17.1 (cont.): Fatigue Design Parameters				
Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding.	C	44×10^8	69	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150×10^{10} (Eqn. 10.17.3.2)	55	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	31	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^{10} (Eqn. 10.17.3.2)	55	At <i>faying surface</i>
8.5 Not fully tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E'	3.9×10^8	48	At the root of the threads extending into the tensile stress area

TABLE 10.17.1 (Cont.) Fatigue Design Parameters	
Illustrative Typical Examples	
SECTION 8 – MISCELLANEOUS	
8.1	
8.2	
8.3	
8.4	
8.5	

10.18 Structural Design for Fire Conditions

This section provides criteria for the design and evaluation of *structural steel* components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and *stiffness* of structural components and systems at elevated temperatures.

10.18.1 General Provisions

The methods contained in this section provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

The section uses the following terms in addition to the terms in the Glossary.

Active fire protection: Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action to mitigate adverse effects.

Compartmentation: The enclosure of a building space with elements that have a specific fire endurance.

Convective heat transfer: The transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.

Design-basis fire: A set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Elevated temperatures: Heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.

Fire: Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.

Fire barrier: Element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.

Fire endurance: A measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.

Fire resistance: That property of assemblies that prevents or retards the passage of excessive heat, hot gases or flames under conditions of use and enables them to continue to perform a stipulated function.

Fire resistance rating: The period of time a building element, component or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.

Flashover: The rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Heat flux: Radiant energy per unit surface area.

Heat release rate: The rate at which thermal energy is generated by a burning material.

Passive fire protection: Building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.

Performance-based design: An engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis and quantitative assessment of alternatives against those design goals and objectives using accepted engineering tools, methodologies and performance criteria.

Prescriptive design: A design method that documents compliance with general criteria established in a building code.

Restrained construction: Floor and roof assemblies and individual beams in buildings where the surrounding or supporting structure is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures.

Unrestrained construction: Floor and roof assemblies and individual beams in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

10.18.1.1 Performance Objective

Structural components, members and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy. Deformation criteria shall be applied where the means of providing structural fire

resistance, or the design criteria for fire barriers, requires consideration of the deformation of the load-carrying structure. Within the compartment of fire origin, forces and deformations from the design basis fire shall not cause a breach of horizontal or vertical compartmentation.

10.18.1.2 Design by Engineering Analysis

The analysis methods in Section 10.18.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 10.18.2 provide evidence of compliance with performance objectives established in Section 10.18.1.1.

The analysis methods in Section 10.18.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the building code.

10.18.1.3 Design by Qualification Testing

The qualification testing methods in Section 10.18.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by building codes.

10.18.1.4 Load Combinations and Required Strength

The *required strength* of the structure and its elements shall be determined from the following *gravity load combination*:

$$[0.9 \text{ or } 1.2]D + T + 0.5L + 0.2S \quad (10.18.1.1)$$

where

D = nominal dead load

L = nominal occupancy live load

S = nominal snow load

T = nominal forces and deformations due to the design-basis fire defined in Section 4.2.1

A lateral *notional load*, $N_i = 0.002Y_i$, as defined in Section 10.20, where N_i = notional *lateral load* applied at framing level i and Y_i = *gravity load* from combination 10.18.1.1 acting on framing level i , shall be applied in combination with the loads stipulated in Equation 10.18.1.1. Unless otherwise stipulated by the *authority having jurisdiction*, D , L and S shall be the *nominal loads* specified in Chapter 2 of Part 6 of this code.

10.18.2 Structural Design for Fire Conditions By Analysis

It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.

10.18.2.1 Design-Basis Fire

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel *load* density based on the occupancy of the space shall be considered when determining the total fuel *load*. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods in Section 10.18.2 are used to demonstrate an equivalency as an alternative material or method as permitted by a building code, the design-basis fire shall be determined in accordance with ASTM E119.

10.18.2.1.1 Localized Fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

10.18.2.1.2 Post-Flashover Compartment Fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include

fuel *load*, ventilation characteristics to the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

10.18.2.1.3 Exterior Fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 10.18.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

10.18.2.1.4 Fire Duration

The fire duration in a particular area shall be determined by considering the total combustible mass, in other words, fuel *load* available in the space. In the case of either a localized fire or a post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Section 10.18.2.1.2.

10.18.2.1.5 Active Fire Protection Systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

10.18.2.2 Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

Table 10.18.2.1 Properties of Steel at Elevated Temperatures			
Steel Temperature [°C]	$k_E = E_m / E$	$k_y = F_{ym} / F_y$	$k_u = F_{um} / F_y$
20	*	*	*
93	1.00	*	*
204	0.90	*	*
316	0.78	*	*
399	0.70	1.00	1.00
427	0.67	0.94	0.94
538	0.49	0.66	0.66
649	0.22	0.35	0.35
760	0.11	0.16	0.16
871	0.07	0.07	0.07
982	0.05	0.04	0.04
1093	0.02	0.02	0.02
1204	0.00	0.00	0.00
*Use ambient properties.			

10.18.2.3 Material Strengths at Elevated Temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with a *yield strength* in excess of 448 MPa or concretes with specified compression strength in excess of 55 MPa.

10.18.2.3.1 Thermal Elongation

Thermal expansion of structural and reinforcing steels: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be $1.4 \times 10^{-5} / ^\circ\text{C}$.

Thermal expansion of normal weight concrete: For calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be $1.8 \times 10^{-5} / ^\circ\text{C}$.

Thermal expansion of lightweight concrete: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be $7.9 \times 10^{-6} / ^\circ\text{C}$.

10.18.2.3.2 Mechanical Properties at Elevated Temperatures

The deterioration in strength and *stiffness* of structural members, components, and systems shall be taken into account in the *structural analysis* of the frame.

The values F_{ym} , F_{um} , E_m , f'_{cm} , E_{cm} and ϵ_{cu} at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, assumed to be 20° C, shall be defined as in Tables 10.18.2.1 and 10.18.2.2. It is permitted to interpolate between these values.

Table 10.18.2.2 Properties of Concrete at Elevated Temperatures				
Concrete Temperature [°C]	$k_c = f'_{cm} / f'_c$		E_{cm} / E_c	$\epsilon_{cu}(\%)$
	NWC	LWC		LWC
20	1.00	1.00	1.00	0.25
93	0.95	1.00	0.93	0.34
204	0.90	1.00	0.75	0.46
288	0.86	1.00	0.61	0.58
316	0.83	0.98	0.57	0.62
427	0.71	0.85	0.38	0.80
538	0.54	0.71	0.20	1.06
649	0.38	0.58	0.092	1.32
760	0.21	0.45	0.073	1.43
871	0.10	0.31	0.055	1.49
982	0.05	0.18	0.036	1.50
1093	0.01	0.05	0.018	1.50
1204	0.00	0.00	0.00	—

10.18.2.4 Structural Design Requirements

10.18.2.4.1 General Structural Integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The *structural system* shall be designed to sustain local damage with the structural system as a whole remaining stable.

Continuous *load* paths shall be provided to transfer all *forces* from the ex- posed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

10.18.2.4.2 Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall develop the strength of the connected members or the forces indicated above. Where the means of providing fire resistance requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

10.18.2.4.3 Methods of Analysis

10.18.2.4.3a Advanced Methods of Analysis

The methods of analysis in this section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 10.18.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The *thermal response* shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature- dependent thermal properties of the structural elements and fire-resistive materials as per Section 10.18.2.2.

The *mechanical response* results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and *stiffness* with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 10.18.2.3.

The resulting analysis shall consider all relevant *limit states*, such as excessive deflections, connection fractures, and overall or *local buckling*.

10.18.2.4.3a Advanced Methods of Analysis

The methods of analysis in this section are applicable for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

(1) Tension members

It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 10.18.2.1.

The *design strength* of a tension member shall be determined using the provisions of Section 10.4, with steel properties as stipulated in Section 10.18.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(2) Compression members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 10.18.2.1.

The design strength of a compression member shall be determined using the provisions of Section 10.5 with steel properties as stipulated in Section 10.18.2.3.

(3) Flexural members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member. The design strength of a flexural member shall be determined using the provisions of Section 10.6 with steel properties as stipulated in Section 10.18.2.3.

(4) Composite floor members

It is permitted to model the thermal response of flexural elements supporting a concrete slab using a one-dimensional heat transfer equation to calculate bottom flange temperature. That temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25 percent from the mid-depth of the web to the top flange of the *beam*.

The design strength of a *composite* flexural member shall be determined using the provisions of Chapter 13 of Part 6 of this code, with reduced *yield stresses* in the steel consistent with the temperature variation described under thermal response.

10.18.2.4.4 Design Strength

The design strength shall be determined as in Section 10.2.3.3. The *nominal strength*, R_n , shall be calculated using material properties, as stipulated in Section 10.18.2.3, at the temperature developed by the design-basis fire.

10.18.3 Design By Qualification Testing

10.18.3.1 Design Strength

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. It shall be permitted to demonstrate compliance with these requirements using the procedures specified for steel construction in Section 5 of ASCE/SFPE 29.

10.18.3.2 Restrained Construction

For floor and roof assemblies and individual *beams* in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting actions caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members (in other words, *columns*, girders) shall be considered restrained construction.

10.18.3.3 Unrestrained Construction

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist actions caused by thermal expansion.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

10.19 Stability Bracing For Columns and Beams

This section addresses the minimum brace strength and *stiffness* necessary to provide member *strengths* based on the *unbraced length* between braces with an *effective length factor*, K , equal to 1.0.

10.19.1 General Provisions

Bracing is assumed to be perpendicular to the members to be braced; for inclined or diagonal bracing, the brace strength (force or moment) and stiffness (force per unit displacement or moment per unit rotation) shall be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of connections and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A relative brace controls the movement of the brace point with respect to adjacent braced points. A nodal brace controls the movement at the braced point without direct interaction with adjacent braced points. The available strength and stiffness of the bracing shall equal or exceed the required limits unless analysis indicates that smaller values are justified by analysis.

A second-order analysis that includes an initial out-of-straightness of the member to obtain brace strength and stiffness is permitted in lieu of the requirements of this section.

10.19.2 Columns

It is permitted to brace an individual column at end and intermediate points along its length by either relative or nodal bracing systems. It is assumed that nodal braces are equally spaced along the column.

10.19.2.1 Relative Bracing

The required brace strength is

$$P_{br} = 0.004 P_r \quad (10.19.2.1)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{2P_r}{L_b} \right) (LRFD) \quad \beta_{br} = \Omega \left(\frac{2P_r}{L_b} \right) (ASD) \quad (10.19.2.2)$$

where

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

For design according to Section 10.2.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

For design according to Section 10.2.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

10.19.2.2 Nodal Bracing

The required brace strength is

$$P_{br} = 0.01 P_r \quad (10.19.2.3)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{8P_r}{L_b} \right) (LRFD) \quad \beta_{br} = \Omega \left(\frac{8P_r}{L_b} \right) (ASD) \quad (10.19.2.4)$$

where

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

For design according to Section 10.2.3.3 (LRFD)

P_r = required axial compressive strength using LRFD load combinations, N

For design according to Section 10.2.3.4 (ASD)

P_r = required axial compressive strength using ASD load combinations, N

When L_b is less than L_q , where L_q is the maximum unbraced length for the required column force with K equal to 1.0, then L_b in Equation 10.19.2.4 is permitted to be taken equal to L_q .

10.19.3 Beams

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided. Beam bracing shall prevent the relative displacement of the top and bottom flanges, in other words, twist of the section. Lateral stability of beams shall be provided by lateral bracing, torsional bracing or a combination of the two. In members subjected to double curvature bending, the inflection point shall not be considered a brace point.

10.19.3.1 Lateral Bracing

Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point nearest the inflection point for beams subjected to double curvature bending along the length to be braced.

a. Relative Bracing

The required brace strength is

$$P_{br} = 0.008 M_r C_d / h_o \quad (10.19.3.1)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{4M_r C_d}{L_b h_o} \right) (LRFD) \quad \beta_{br} = \Omega \left(\frac{4M_r C_d}{L_b h_o} \right) (ASD) \quad (10.19.3.2)$$

Where,

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

h_o = distance between flange centroids, mm.

C_d = 1.0 for bending in *single curvature*; 2.0 for double curvature; C_d = 2.0 only applies to the brace closest to the inflection point.

L_b = laterally unbraced length, mm.

For design according to Section 10.2.3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Section 10.2.3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, N-mm

b. Nodal Bracing

The required brace strength is

$$P_{br} = 0.02 M_r C_d / h_o \quad (10.19.3.3)$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\phi} \left(\frac{10 M_r C_d}{L_b h_o} \right) (LRFD) \quad \beta_{br} = \Omega \left(\frac{10 M_r C_d}{L_b h_o} \right) (ASD) \quad (10.19.3.4)$$

where

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$$

For design according to Section 10.2.3.3 (LRFD)

M_r = required flexural strength using *LRFD load combinations*, N-mm

For design according to Section 10.2.3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, N-mm

When L_b is less than L_q , the maximum unbraced length for M_r , then L_b in Equation 10.19.3.4 shall be permitted to be taken equal to L_q .

10.19.3.2 Torsional Bracing

It is permitted to provide either nodal or continuous *torsional bracing* along the *beam* length. It is permitted to attach the bracing at any cross-sectional location and it need not be attached near the compression flange. The *connection* between a torsional brace and the beam shall be able to support the required moment given below.

a. Nodal Bracing

The required bracing moment is

$$M_{br} = \frac{0.024 M_r L}{n C_b L_b} \quad (10.19.3.5)$$

The required cross-frame or diaphragm bracing stiffness is

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \quad (10.19.3.6)$$

where

$$\beta_T = \frac{1}{\phi} \left(\frac{2.4 L M_r^2}{n E I_y C_b^2} \right) (LRFD) \quad \beta_T = \Omega \left(\frac{2.4 L M_r^2}{n E I_y C_b^2} \right) (ASD) \quad (10.19.3.7)$$

$$\beta_{sec} = \frac{3.3 E}{h_o} \left(\frac{1.5 h_o t_w^3}{12} + \frac{t_s b_s^3}{12} \right) \quad (10.19.3.8)$$

where

$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 3.00 \text{ (ASD)}$$

L = span length, mm

n = number of nodal braced points within the span

E = modulus of elasticity of steel 200 000 MPa

I_y = out-of-plane moment of inertia, mm⁴

C_b = modification factor defined in Section 10.6

t_w = beam web thickness, mm

t_s = web stiffener thickness, mm

b_s = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), mm.

β_T = brace stiffness excluding web distortion, N-mm/radian

β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners,

if any, N-mm/radian

For design according to Section 10.2.3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, N-mm

For design according to Section 10.2.3.4 (ASD)

M_r = required flexural strength using ASD load combinations, N-mm

If $\beta_{sec} < \beta_T$, Equation 10.19.3.6 is negative, which indicates that torsional beam bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any beam flange that is not directly attached to the torsional brace. When L_b is less than L_q , then L_b in Equation 10.19.3.5 shall be permitted to be taken equal to L_q .

b. Continuous Torsional Bracing

For continuous bracing, use Equations 10.19.3.5, 10.19.3.6 and 10.19.3.8 with L/n taken as 1.0 and L_b taken as L_q ; the bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3 E t_w^3}{12 h_o} \quad (10.19.3.9)$$

10.19.4 Slenderness Limitations

The slenderness ratio, L/r , of any stability bracing shall not exceed 180 unless a comprehensive analysis including second order effects justifies a higher value.

10.20 Seismic Provisions for Structural Steel Buildings

10.20.1 Scope

The *Seismic Provisions for Structural Steel Buildings*, hereinafter referred to as these *Provisions* as outline in this Section 10.20, shall govern the design, fabrication and erection of structural steel members and connections in the *seismic load resisting systems* (SLRS) and splices in columns that are not part of the SLRS, in buildings and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting-elements.

These *Provisions* shall apply when the *seismic response modification coefficient*, R , (as specified in Chapter 2 of Part 6) is taken greater than 3, regardless of the *seismic design category*. When the seismic response modification coefficient, R , is taken as 3 or less, the structure is not required to satisfy the *Provisions* this Section 10.20, unless specifically required by the applicable authority

These *Provisions* shall be applied in conjunction with the specification set forth in Sections 10.1 through 10.19 whichever is applicable.

Loads, load combinations, system limitations and general design requirements shall be those in Chapter 2 of Part 6 of this code as well as those mentioned in Section 10.2.

10.20.2 Referenced Specifications, Codes and Standards

The documents referenced in these *Provisions* shall include those listed in Section 10.1.2 with the following additions and modifications:

American Institute of Steel Construction (AISC):

Specification for Structural Steel Buildings, ANSI/AISC 360-05

Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, ANSI/AISC 358-05

American Society for Nondestructive Testing (ASNT):

Recommended Practice for the Training and Testing of Nondestructive Testing Personnel, ASNT SNT TC-1a-2001

Standard for the Qualification and Certification of Nondestructive Testing Personnel, ANSI/ASNT CP-189-2001

American Welding Society (AWS):

Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding, AWS A4.3-93R

Standard Methods for Mechanical Testing of Welds-U.S. Customary, ANSI/ AWS B4.0-98

Standard Methods for Mechanical Testing of Welds—Metric Only, ANSI/AWS B4.0M:2000

Standard for the Qualification of Welding Inspectors, AWS B5.1:2003

Oxygen Cutting Surface Roughness Gauge and Wall Chart for Criteria Describing Oxygen-Cut Surfaces, AWS C4.1

Federal Emergency Management Agency (FEMA)

Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, FEMA 350, July 2000

Symbols

Symbols used in this section are listed below. Numbers in parentheses after the definition refer to the section in these Provisions in which the symbol is first used.

<u>Symbol</u>	<u>Description</u>	<u>Section</u>
A_b	Cross-sectional area of a horizontal boundary element (HBE), (mm^2)	10.20.17.2.1
A_c	Cross-sectional area of a vertical boundary element (VBE), (mm^2)	10.20.17.2.1
A_f	Flange area, (mm^2)	10.20.8
A_g	Gross area, (mm^2)	10.20.9
A_{sc}	Area of the yielding segment of steel core, (mm^2)	10.20.16
A_{st}	Area of link stiffener, (mm^2)	10.20.15
A_w	Link web area, (mm^2)	10.20.15
C_a	Ratio of required strength to available strength	Table 10.20.8.1
C_d	Coefficient relating relative brace stiffness and curvature	10.20.9
C_d	Deflection amplification factor	10.C.2
C_r	Parameter used for determining the approximate fundamental period	10.C.2
D	Dead load due to the weight of the structural elements and permanent features on the building, (N)	10.20.9
D	Outside diameter of round HSS, (mm)	Table 10.20.8.1
E	Earthquake load	10.20.4
E	Effect of horizontal and vertical earthquake-induced loads	10.20.9
E	Modulus of elasticity of steel, $E = 200,000$ MPa	10.20.8
EI	Flexural elastic stiffness of the chord members of the special segment, (N-mm ²)	10.20.12
F_y	Specified minimum yield stress of the type of steel to be used, (MPa). As used in the <i>Specification</i> , “yield stress” denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have a yield point)	10.20.6
F_{yb}	F_y of a beam, (MPa)	10.20.9

Symbol	Description	Section
F_{yc}	F_y of a column, (MPa)	10.20.9
F_{ysc}	Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, (MPa) .	10.20.16
F_u	Specified minimum tensile strength, (MPa)	10.20.6
H	Height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, (mm)	10.20.8
I	Moment of inertia, (mm ⁴)	10.20.12
I_c	Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, (mm ⁴)	10.20.17
K	Effective length factor for prismatic member	10.20.13
L	Span length of the truss, (mm)	10.20.12
L	Distance between VBE centerlines, (mm)	10.20.17
L_b	Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, (mm)	10.20.13
L_b	Link length, (mm)	10.20.15
L_{cf}	Clear distance between VBE flanges, (mm)	10.20.17
L_h	Distance between plastic hinge locations, (mm)	10.20.9
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, (mm)	10.20.12
L_{pd}	Limiting laterally unbraced length for plastic analysis, (mm)	10.20.13
L_s	Length of the special segment, (mm)	10.20.12
M_a	Required flexural strength, using ASD load combinations, (N-mm)	10.20.9
M_{av}	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on ASD load combinations, (N-mm)	10.20.9
M_n	Nominal flexural strength, (N-mm)	10.20.11
M_{nc}	Nominal flexural strength of the chord member of the special segment, (N-mm)	10.20.12
M_p	Nominal plastic flexural strength, (N-mm)	Table 10.20.8.1
M_{pa}	Nominal plastic flexural strength modified by axial load, (N-mm)	10.20.15
M_{pb}	Nominal plastic flexural strength of the beam, (N-mm)	10.20.9
$M_{p,exp}$	Expected plastic moment, (N-mm)	10.20.9
M_{pc}	Nominal plastic flexural strength of the column, (N-mm)	10.20.8
M_r	Expected flexural strength, (N-mm)	10.20.9
M_{uv}	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD load combinations, (N-mm)	10.20.9
M_u	Required flexural strength, using LRFD load combinations, (N-mm)	10.20.9
$M_{u,exp}$	Expected required flexural strength, (N-mm)	10.20.15

Symbol	Description	Section
P_a	Required axial strength of a column using ASD load combinations, (N)	10.20.8
P_{ac}	Required compressive strength using ASD load combinations, (N)	10.20.9
P_b	Required strength of lateral brace at ends of the link, (N)	10.20.15
P_c	Available axial strength of a column, (N)	10.20.9
P_n	Nominal axial strength of a column, (N)	10.20.8
P_{nc}	Nominal axial compressive strength of diagonal members of the special segment, (N)	10.20.12
P_{nt}	Nominal axial tensile strength of diagonal members of the special segment, (N)	10.20.12
P_r	Required compressive strength, (N)	10.20.15
P_{rc}	Required compressive strength using ASD or LRFD load combinations, (N)	10.20.9
P_u	Required axial strength of a column or a link using LRFD load combinations, (N)	10.20.8
P_{uc}	Required compressive strength using LRFD load combinations, (N)	10.20.9
P_y	Nominal axial yield strength of a member, equal to $F_y A_g$, (N)	Table 10.20.8.1
P_{ysc}	Axial yield strength of steel core, (N)	10.20.16
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces, (N)	10.20.13
Q_1	Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link	10.20.15
R	Seismic response modification coefficient	10.20.1
R_n	Nominal strength, (N)	10.20.6
R_t	Ratio of the expected tensile strength to the specified minimum tensile strength F_u , as related to overstrength in material yield stress R_y	10.20.6
R_u	Required strength	10.20.9
R_v	Panel zone nominal shear strength	10.20.9
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y	10.20.6
S	Snow load, (N)	10.20.9
V_a	Required shear strength using ASD load combinations, (N)	10.20.9
V_n	Nominal shear strength of a member, (N)	10.20.15
V_{ne}	Expected vertical shear strength of the special segment, (N)	10.20.12
V_p	Nominal shear strength of an active link, (N)	Table 10.20.8.1
V_{pa}	Nominal shear strength of an active link modified by the axial load magnitude, (N)	10.20.15
V_u	Required shear strength using LRFD load combinations, (N)	10.20.10
Z	Plastic section modulus of a member, (mm^3)	10.20.9
Z_b	Plastic section modulus of the beam, (mm^3)	10.20.9
Z_c	Plastic section modulus of the column, (mm^3)	10.20.9

Symbol	Description	Section
Z_x	Plastic section modulus x-axis, (mm^3)	10.20.8
Z_{RBS}	Minimum plastic section modulus at the reduced beam section, (mm^3)	10.20.9
α	Angle that diagonal members make with the horizontal	10.20.12
b	Width of compression element as defined in <i>Specification</i> Section 10.2.4.1, (mm)	Table 10.20.8.1
b_{cf}	Width of column flange, (mm)	10.20.9
b_f	Flange width, (mm)	10.20.9
d	Nominal fastener diameter, (mm)	10.20.7
d	Overall beam depth, (mm)	10.20.15
d_c	Overall column depth, (mm)	10.20.9
d_z	Overall panel zone depth between continuity plates, (mm)	10.20.9
e	EBF link length, (mm)	10.20.15
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; and for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, (mm)	Table 10.20.8.1
h	Distance between horizontal boundary element centerlines, (mm)	10.20.17
h_o	Distance between flange centroids, (mm)	10.20.9
l	Unbraced length between stitches of built-up bracing members, (mm)	10.20.13
l	Unbraced length of compression or bracing member, (mm)	10.20.13
r	Governing radius of gyration, (mm)	10.20.13
r_y	Radius of gyration about y-axis, (mm)	10.20.9
t	Thickness of connected part, (mm)	10.20.7
t	Thickness of element, (mm)	Table 10.20.8.1
t	Thickness of column web or doubler plate, (mm)	10.20.9
t_{bf}	Thickness of beam flange, (mm)	10.20.9
t_{cf}	Thickness of column flange, (mm)	10.20.9
t_f	Thickness of flange, (mm)	10.20.17
t_p	Thickness of panel zone including doubler plates, (mm)	10.20.9
t_w	Thickness of web, (mm)	Table 10.20.8.1
w_z	Width of panel zone between column flanges, (mm)	10.20.9
x	Parameter used for determining the approximate fundamental period	10.C.2
z_b	Minimum plastic section modulus at the reduced beam section, (mm^3)	10.20.9
ΣM_{pc}^*	Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g , from the top and bottom of the beam moment connection	10.20.9
ΣM_{pb}^*	Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum	10.20.9

Symbol	Description	Section
	developed moments from the column face. Maximum developed moments shall be determined from test results	
β	Compression strength adjustment factor	10.20.16
Δ	Design story drift	10.20.15
Δ_b	Deformation quantity used to control loading of test specimen (total brace end rotation for the subassemblage test specimen; total brace axial deformation for the brace test specimen)	10.E.2
Δ_{bm}	Value of deformation quantity, Δ_b , corresponding to the design story drift	10.E.6
Δ_{by}	Value of deformation quantity, Δ_b , at first significant yield of test specimen	10.E.6
Ω	Safety factor	10.20.6
Ω_b	Safety factor for flexure = 1.67	10.20.8
Ω_c	Safety factor for compression = 1.67	10.20.8
Ω_o	Horizontal seismic overstrength factor	10.20.4
Ω_v	Safety factor for shear strength of panel zone of beam-to-column connections	10.20.9
α	Angle of diagonal members with the horizontal	10.20.12
α	Angle of web yielding in radians, as measured relative to the vertical	10.20.17
δ	Deformation quantity used to control loading of test specimen	10.D.6
δ_y	Value of deformation quantity δ at first significant yield of test specimen	10.D.6
$\rho_{\square}$	Ratio of required axial force P_u to required shear strength V_u of a link	10.20.15
λ_p, λ_{ps}	Limiting slenderness parameter for compact element	10.20.8
ϕ	Resistance factor	10.20.6
ϕ_b	Resistance factor for flexure	10.20.8
ϕ_c	Resistance factor for compression	10.20.8
ϕ_v	Resistance factor for shear strength of panel zone of beam-to-column connections	10.20.9
ϕ_v	Resistance factor for shear	10.20.15
θ	Interstory drift angle, radians	10.D.3
γ_{total}	Link rotation angle	10.D.2
ω	Strain hardening adjustment factor	10.20.16

Glossary

Notes:

(1) Terms designated with * are usually qualified by the type of *load effect*, for example, *nominal tensile strength*, *available compressive strength*, *design flexural strength*.

(2) Terms designated with ** are usually qualified by the type of component, for example, *web local buckling*, *flange local bending*.

Adjusted brace strength. Strength of a brace in a *buckling-restrained braced frame* at deformations corresponding to 2.0 times the *design story drift*.

*Allowable strength**. Nominal strength divided by the safety factor, R_n / Ω .

Applicable building code (ABC). Building code under which the structure is designed.

Amplified seismic load. Horizontal component of earthquake load E multiplied by Ω_e , where E and the horizontal component of E are specified in the *applicable building code*.

Authority having jurisdiction (AHJ). Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this standard.

*Available strength**. Design strength or allowable strength, as appropriate.

ASD (Allowable Strength Design). Method of proportioning structural components such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the ASD load combinations.

ASD load combination. Load combination in the *applicable building code* intended for allowable strength design (allowable stress design).

Buckling-restrained braced frame (BRBF). Diagonally braced frame satisfying the requirements of Section 16 in which all members of the bracing system are subjected primarily to axial forces and in which the limit state of compression buckling of braces is precluded at forces and deformations corresponding to 2.0 times the *design story drift*.

Buckling-restraining system. System of restraints that limits buckling of the steel core in BRBF. This system includes the casing on the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the *design story drift*.

Casing. Element that resists forces transverse to the axis of the brace thereby restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force in the axis of the brace.

Column base. Assemblage of plates, connectors, bolts, and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

Continuity plates. Column stiffeners at the top and bottom of the *panel zone*; also known as transverse stiffeners.

Contractor. Fabricator or erector, as applicable.

Demand critical weld. Weld so designated by these *Provisions*.

Design earthquake. The earthquake represented by the *design response spectrum* as specified in the *applicable building code*.

Design story drift. Amplified story drift (drift under the *design earthquake*, including the effects of inelastic action), determined as specified in the *applicable building code*.

*Design strength**. Resistance factor multiplied by the *nominal strength*, ϕR_n .

Diagonal bracing. Inclined structural members carrying primarily axial load that are employed to enable a structural frame to act as a truss to resist lateral loads.

Dual system. Structural system with the following features: (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment frames (SMF, IMF or OMF) that are capable of resisting at least 25 percent of the base shear, and concrete or steel shear walls, or steel braced frames (EBF, SCBF or OCBF); and (3) each system designed to resist the total lateral load in proportion to its relative rigidity.

Ductile limit state. Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width-thickness limitations of Table I-8-1. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.

Eccentrically braced frame (EBF). Diagonally braced frame meeting the requirements of Section 15 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection.

Exempted column. Column not meeting the requirements of Equation 9-3 for SMF.

Expected yield strength. Yield strength in tension of a member, equal to the expected yield stress multiplied by A_g .

*Expected tensile strength **. Tensile strength of a member, equal to the specified minimum tensile strength, F_u , multiplied by R_t .

Expected yield stress. Yield stress of the material, equal to the specified minimum yield stress, F_y , multiplied by R_y .

Intermediate moment frame (IMF). Moment frame system that meets the requirements of Section 10.20.10

Interstory drift angle. Interstory displacement divided by story height, radians.

Inverted-V-braced frame. See *V-braced frame*.

k-area. The *k-area* is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC "k" dimension) a distance of 38 mm into the web beyond the "k" dimension.

K-braced frame. A bracing configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.

Lateral bracing member. Member that is designed to inhibit lateral buckling or lateral-torsional buckling of primary framing members.

Link. In EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column. The length of the *link* is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.

Link intermediate web stiffeners. Vertical web stiffeners placed within the link in EBF.

Link rotation angle. Inelastic angle between the *link* and the beam outside of the link when the total story drift is equal to the *design story drift*.

Link shear design strength. Lesser of the available shear strength of the *link* developed from the moment or shear strength of the link.

Lowest Anticipated Service Temperature (LAST). The lowest 1-hour average temperature with a 100-year mean recurrence interval.

LRFD (Load and Resistance Factor Design). Method of proportioning structural components such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations*.

LRFD Load Combination. Load combination in the *applicable building code* intended for strength design (*load and resistance factor design*).

Measured flexural resistance. Bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Appendix S.

Nominal load. Magnitude of the *load* specified by the *applicable building code*.

Nominal strength.* Strength of a structure or component (without the *resistance factor* or *safety factor* applied) to resist the load effects, as determined in accordance with this *Specification*.

Ordinary concentrically braced frame (OCBF). Diagonally braced frame meeting the requirements of Section 14 in which all members of the bracing system are subjected primarily to axial forces.

Ordinary moment frame (OMF). Moment frame system that meets the requirements of Section 10.20.11.

Overstrength factor, Ω_o . Factor specified by the *applicable building code* in order to determine the amplified seismic load, where required by these *Provisions*.

Prequalified connection. Connection that complies with the requirements of Appendix 10.A

Protected zone. Area of members in which limitations apply to fabrication and attachments.

Prototype. The connection or brace design that is to be used in the building (SMF, IMF, EBF, and BRBF).

Quality assurance plan. Written description of qualifications, procedures, quality inspections, resources, and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.

Reduced beam section. Reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.

Required strength.* Forces, stresses, and deformations produced in a structural component, determined by either structural analysis, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by the *Specification* and these *Provisions*.

Resistance factor, ϕ . Factor that accounts for unavoidable deviations of the *nominal strength* from the actual strength and for the manner and consequences of failure.

Safety factor, Ω . Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the *nominal load*, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Seismic design category. Classification assigned to a building by the *applicable building code* based upon its *seismic use group* and the design spectral response acceleration coefficients.

Seismic load resisting system (SLRS). Assembly of structural elements in the building that resists seismic loads, including struts, collectors, chords, diaphragms and trusses.

Seismic response modification coefficient, R . Factor that reduces seismic load effects to strength level as specified by the *applicable building code*.

Seismic use group. Classification assigned to a structure based on its use as specified by the *applicable building code*.

Special concentrically braced frame (SCBF). Diagonally braced frame meeting the requirements of Section 10.20.13 in which all members of the bracing system are subjected primarily to axial forces.

Special moment frame (SMF). Moment frame system that meets the requirements of Section 10.20.9.

Special plate shear wall (SPSW). Plate shear wall system that meets the requirements of Section 10.20.17.

Special truss moment frame (STMF). Truss moment frame system that meets the requirements of Section 10.20.12.

Static yield strength. Strength of a structural member or connection determined on the basis of testing conducted under slow monotonic loading until failure.

Steel core. Axial-force-resisting element of braces in BRBF. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.

Tested connection. Connection that complies with the requirements of Appendix 10.B.

V-braced frame. Concentrically braced frame (SCBF, OCBF or BRBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an *inverted-V-braced frame*.

X-braced frame. Concentrically braced frame (OCBF or SCBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

Y-braced frame. Eccentrically braced frame (EBF) in which the stem of the Y is the *link* of the EBF system.

10.20.3 General Seismic Design Requirements

The required *strength* and other seismic provisions and the limitations on height and irregularity are specified in Chapter 2 of Part 6 of this code.

The design *story drift* shall be in accordance with the requirements set forth in Chapter 2 of Part 6 of this code.

10.20.4 Loads, Load Combinations, and Nominal Strengths

10.20.4.1 Loads and Load Combinations

The loads and load combinations shall be as stipulated in Chapter 2 of Part 6 of this code. Where *amplified seismic loads* are required by these *Provisions*, the horizontal portion of the earthquake load *E* (as defined in Chapter 2 of Part 6) shall be multiplied by the overstrength factor, Ω_o , as applicable.

10.20.4.2 Nominal Strength

The *nominal strength* of systems, members and connections shall comply with the *Specification*, except as modified throughout these *Provisions*.

10.20.5 Structural Design Drawings and Specifications, Shop Drawings, and Erection Drawings

10.20.5.1 Structural Design Drawings and Specifications

Structural design drawings and specifications shall show the work to be performed, and include items required by the *Specification* and the following, as applicable:

- (1) Designation of the seismic load resisting system (SLRS)
- (2) Designation of the members and connections that are part of the SLRS
- (3) Configuration of the connections
- (4) Connection material specifications and sizes
- (5) Locations of demand critical welds
- (6) Lowest anticipated service temperature (LAST) of the steel structure, if the structure is not enclosed and maintained at a temperature of 10 °C or higher.
- (7) Locations and dimensions of protected zones
- (8) Locations where gusset plates are to be detailed to accommodate inelastic rotation
- (9) Welding requirements as specified in Appendix 10.F, Section 10.F.2.1.

Shop Drawings

Shop drawings shall include items required by the *Specification* and the following, as applicable:

- (1) Designation of the members and connections that are part of the SLRS
- (2) Connection material specifications
- (3) Locations of *demand critical* shop welds

(4) Locations and dimensions of *protected zones*

(5) Gusset plates drawn to scale when they are detailed to accommodate inelastic rotation

Welding requirements as specified in Appendix 10.F, Section 10.F.2.2.

10.20.5.2 Erection Drawings

Erection drawings shall include items required by the *Specification* and the following, as applicable:

(1) Designation of the members and connections that are part of the SLRS

(2) Field connection material specifications and sizes

(3) Locations of *demand critical* field welds

(4) Locations and dimensions of *protected zones*

(5) Locations of pretensioned bolts

(6) Field welding requirements as specified in Appendix 10.F, Section 10.F.2.3

10.20.6 Materials

10.20.6.1 Material Specifications

Structural steel used in the seismic *load resisting system* (SLRS) shall meet the requirements of Section 10.1.3.1a, except as modified in present Section 10.20. The specified minimum yield stress of steel to be used for members in which inelastic behavior is expected shall not exceed 345 MPa for systems defined in Sections 10.20.9, 10.20.10, 10.20.12, 10.20.13, 10.20.15, 10.20.16, and 10.20.17 nor 380 MPa for systems defined in Sections 10.20.11 and 10.20.14, unless the suitability of the material is determined by testing or other rational criteria. This limitation does not apply to columns for which the only expected inelastic behavior is yielding at the *column base*.

The structural steel used in the SLRS described in Sections 10.20.9 through 10.20.17 shall meet one of the following ASTM Specifications: A36/ A36M, A53/A53M, A500 (Grade B or C), A501, A529/A529M, A572/A572M [Grade 290, 345 or 380], A588/A588M, A913/A913M [Grade 345, 415 or 450], A992/A992M, or A1011 HSLAS Grade 380. The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D.

Other steels and non-steel materials in *buckling-restrained braced frames* are permitted to be used subject to the requirements of Section 10.20.16 and Appendix 10.E.

Material Properties for Determination of Required Strength of Members and Connections

The required strength of an element (a member or a connection) shall be determined from the *expected yield stress*, $R_y F_y$, of an adjoining member, where F_y is the specified minimum yield stress of the grade of steel to be used in the adjoining members and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y , of that material.

The available strength of the element, ϕR_n for LRFD and R_n / Ω for ASD, shall be equal to or greater than the required strength, where R_n is the nominal strength of the connection. The expected tensile strength, $R_t F_u$, and the expected yield stress, $R_y F_y$, are permitted to be used in lieu of F_u and F_y , respectively, in determining the nominal strength, R_n , of rupture and yielding limit states within the same member for which the required strength is determined.

The values of R_y and R_t for various steels are given in Table 10.20.1. Other values of R_y and R_t shall be permitted if the values are determined by testing of specimens similar in size and source conducted in accordance with the requirements for the specified grade of steel.

10.20.6.2 Heavy Section CVN Requirements

For structural steel in the SLRS, in addition to the requirements of Section 10.1.3.1c, hot rolled shapes with flange thickness 38 mm and thicker shall have a minimum Charpy V-Notch toughness of 27 J at 21°C, tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates 50 mm thick and thicker shall have a minimum Charpy V-Notch toughness of 27 J at 21 °C, measured at any location permitted by ASTM A673, where the plate is used in the following:

1. Members built-up from plate
2. Connection plates where inelastic strain under seismic loading is expected
3. As the *steel core* of buckling-restrained braces

TABLE 10.20.1: R_y and R_t Values for Different Member Types

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A572/572M Grade 42 (290)	1.3	1.1
• ASTM A572/572M Grade 50 (345) or 55 (380), ASTM A913/A913M Grade 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M, A1011 HSLAS Grade 55 (380)	1.1	1.1
• ASTM A529 Grade 50 (345)	1.2	1.2
• ASTM A529 Grade 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500 (Grade B or C), ASTM A501	1.4	1.3
Pipe:		
• ASTM A53/A53M	1.6	1.2
Plates:		
• ASTM A36/A36M	1.3	1.2
• ASTM A572/A572M Grade 50 (345), ASTM A588/A588M	1.1	1.2

10.20.7 Connections, Joints and Fasteners

10.20.7.1 Scope

Connections, joints and fasteners that are part of the *seismic load resisting system* (SLRS) shall comply with Section 10.10, and with the additional requirements of this Section.

The design of connections for a member that is a part of the SLRS shall be configured such that a *ductile limit state* in either the connection or the member controls the design.

10.20.7.2 Bolted Joints

All bolts shall be pre-tensioned high strength bolts and shall meet the requirements for *slip-critical* faying surfaces in accordance with Section 10.10.3.8 with a Class A surface. Bolts shall be installed in standard holes or in short-slotted holes perpendicular to the applied load. For brace diagonals, oversized holes shall be permitted when the connection is designed as a slip-critical joint, and the oversized hole is in one ply only. Alternative hole types are permitted if determined in a connection prequalification in accordance with Appendix 10A, or if determined in a program of qualification testing in accordance with Appendix 10.D or 10.E. The *available shear strength* of bolted joints using standard holes shall be calculated as that for bearing-type joints in accordance with Sections 10.10.3.7 and 10.10.3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

Exception: The faying surfaces for end plate moment connections are permitted to be coated with coatings not tested for slip resistance, or with coatings with a slip coefficient less than that of a Class A faying surface.

Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

10.20.7.3 Welded Joints

Welding shall be performed in accordance with Appendix 10.F. Welding shall be performed in accordance with a welding procedure specification (WPS) as required in AWS D1.1. The WPS variables shall be within the parameters established by the filler metal manufacturer.

General Requirements

All welds used in members and connections in the SLRS shall be made with a filler metal that can produce welds that have a minimum Charpy V-Notch toughness of 27 J at minus 18 °C, as determined by the appropriate AWS A5 classification test method or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these *Provisions*.

Demand Critical Welds

Where welds are designated as *demand critical*, they shall be made with a filler metal capable of providing a minimum Charpy V-Notch (CVN) toughness of 27 J at 29 °C as determined by the appropriate AWS classification test method or manufacturer certification, and 54 J at 21 °C as determined by Appendix 10.G or other approved method, when the steel frame is normally enclosed and maintained at a temperature of 10 °C or higher. For structures with service temperatures lower than 10 °C, the qualification temperature for Appendix 10.G shall be 11 °C above the *lowest anticipated service temperature*, or at a lower temperature.

SMAW electrodes classified in AWS A5.1 as E7018 or E7018-X, SMAW electrodes classified in AWS A5.5 as E7018-C3L or E8018-C3, and GMAW solid electrodes are exempted from production lot testing when the CVN toughness of the electrode equals or exceeds 27 J at a temperature not exceeding 29 °C as determined by AWS classification test methods. The manufacturer's certificate of compliance shall be considered sufficient evidence of meeting this requirement.

Protected Zone

Where a *protected zone* is designated by these *Provisions*, it shall comply with the following:

- (1) Within the protected zone, discontinuities created by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging and thermal cutting shall be repaired as required by the engineer of record.
- (2) Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on beam flanges within the protected zone. Decking arc spot welds as required to secure decking shall be permitted.
- (3) Welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping or other construction shall not be placed within the protected zone.

Exception: Welded shear studs and other connections shall be permitted when determined in accordance with a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Outside the protected zone, calculations based upon the expected moment shall be made to demonstrate the adequacy of the member net section when connectors that penetrate the member are used.

Continuity Plates and Stiffeners

Corners of *continuity plates* and stiffeners placed in the webs of rolled shapes shall be clipped as described below. Along the web, the clip shall be detailed so that the clip extends a distance of at least 38 mm beyond the published k detail dimension for the rolled shape. Along the flange, the clip shall be detailed so that the clip does not exceed a distance of 12 mm beyond the published k_1 detail dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. If a curved clip is used, it shall have a minimum radius of 12 mm.

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer of record. Unless specified by the engineer of record that they be removed, weld tabs shall not be removed when used in this location.

10.20.8 Members

10.20.8.1 Scope

Members in the seismic load resisting system (SLRS) shall comply with the specifications of Sections 10.1 through 10.11 and Section 10.20.8. For columns that are not part of the SLRS, see Section 10.20.8.4.2.

10.20.8.2 Classification of Sections for Local Buckling

Compact

When required by these Provisions, members of the SLRS shall have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_p , from Specification Table B4.1.

Seismically Compact

When required by these Provisions, members of the SLRS must have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_{ps} , from Provisions Table 10.2.4.1.

Column Strength

When $P_u/\phi_c P_n$ (LRFD) > 0.4 or $\Omega_c P_o/P_n$ (ASD) > 0.4 , as appropriate, without consideration of the amplified seismic load,

where

$$\phi_c = 0.90 \text{ (LRFD)} \quad \Omega_c = 1.67 \text{ (ASD)}$$

P_o = required axial strength of a column using ASD load combinations, N

P_n = nominal axial strength of a column, N

P_u = required axial strength of a column using LRFD load combinations, N

The following requirements shall be met:

The *required axial compressive and tensile strength*, considered in the absence of any applied moment, shall be determined using the load combinations stipulated by the *applicable building code* including the amplified seismic load.

The required axial compressive and tensile strength shall not exceed either of the following:

- The maximum load transferred to the column considering $1.1R_y$ (LRFD) or $(1.1/1.5)R_y$ (ASD), as appropriate, times the *nominal strengths* of the connecting beam or brace elements of the building.
- The limit as determined from the resistance of the foundation to over-turning uplift.

TABLE 10.20.2: Limiting Width-Thickness Ratios for Compression Elements

Description of Element	Width- Thickness Ratio	Limiting Width- Thickness Ratios
		λ_{ps} (seismically compact)
Unstiffened Elements	Flexure in flanges of rolled or built-up I-shaped sections [a], [c], [e], [g], [h]	$0.30\sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [b], [h]	$0.30\sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [d]	$0.38\sqrt{E/F_y}$
	Uniform compression in flanges of channels, outstanding legs of pairs of angles in continuous contact, and braces [e], [e]	$0.30\sqrt{E/F_y}$
	Uniform compression in flanges of H-pile sections	$0.45\sqrt{E/F_y}$
	Flat bars [f]	2.5
	Uniform compression in legs of single angles, legs of double angle members with separators, or flanges of tees [g]	$0.30\sqrt{E/F_y}$
	Uniform compression in stems of tees [g]	$0.30\sqrt{E/F_y}$

Note: See continued Table 10.20.2 for stiffened elements.

TABLE 10.20.2 (cont.)
Limiting Width-Thickness Ratios for
Compression Elements

Description of Element		Width-Thickness Ratio	Limiting Width-Thickness Ratios
			λ_{ps} (seismically compact)
Stiffened Elements	Webs in flexural compression in beams in SMF, Section 10.20.9, unless noted otherwise	h/t_w	$2.45 \sqrt{E/F_y}$
	Webs in flexural compression or combined flexure and axial compression [a], [c], [g], [h], [i], [j]	h/t_w	for $C_a \leq 0.125$ [k] $3.14 \sqrt{\frac{E}{F_y}} (1 - 1.54 C_a)$
			for $C_a > 0.125$ [k] $1.12 \sqrt{\frac{E}{F_y}} (2.33 - C_a) \geq 1.49 \sqrt{\frac{E}{F_y}}$
	Round HSS in axial and/or flexural compression [c], [g]	D/t	$0.044 E/F_y$
	Rectangular HSS in axial and/or flexural compression [c], [g]	b/t or h/t_w	$0.64 \sqrt{E/F_y}$
	Webs of H-Pile sections	h/t_w	$0.94 \sqrt{E/F_y}$
[a] Required for beams in SMF, Section 10.20.9 and SPSW, Section 10.20.17. [b] Required for columns in SMF, Section 10.20.9, unless the ratios from Equation 10.20.9.3 are greater than 2.0 where it is permitted to use λ_p in Specification Table 10.2.4.1. [c] Required for braces and columns in SCBF, Section 10.20.13 and braces in OCBF, Section 10.20.14. [d] It is permitted to use λ_p in Specification Table 10.2.4.1 for columns in STMF, Section 10.20.12 and columns in EBF, Section 10.20.15. [e] Required for link in EBF, Section 10.20.15, except it is permitted to use λ_p in Table 10.2.4.1 of the Specification for flanges of links of length $1.6 M_p / V_p$ or less, where M_p and V_p are defined in Section 10.20.15. [f] Diagonal web members within the special segment of STMF, Section 10.20.12. [g] Chord members of STMF, Section 10.20.12. [h] Required for beams and columns in BRBF, Section 10.20.16. [i] Required for columns in SPSW, Section 10.20.17. [j] For columns in STMF, Section 10.20.12; columns in SMF, if the ratios from Equation 10.20.9.3 are greater than 2.0; columns in EBF, Section 10.20.15; or EBF webs of links of length $1.6 M_p / V_p$ or less, it is permitted to use the following for λ_p: $\text{for } C_a \leq 0.125, \lambda_p = 3.76 \sqrt{\frac{E}{F_y}} (1 - 275 C_a)$ $\text{for } C_a > 0.125, \lambda_p = 1.12 \sqrt{\frac{E}{F_y}} (2.33 - C_a) \geq 1.49 \sqrt{\frac{E}{F_y}}$ [k] For LRFD, $C_a = \frac{P_u}{\phi_b P_y}$ For ASD, $C_a = \frac{\Omega_b P_a}{P_y}$ where P_a = required compressive strength (ASD), N P_u = required compressive strength (LRFD), N P_y = axial yield strength, N $\phi_b = 0.90$ $\Omega_b = 1.67$			

10.20.8.3 Column Splices

General

The *required strength* of column splices in the *seismic load resisting system (SLRS)* shall equal the required strength of the columns, including that determined from Sections 10.20.8.3, 10.20.9.9, 10.20.10.9, 10.20.11.9, 10.20.13.5 and 10.20.16.5.2.

In addition, welded column splices that are subject to a calculated net tensile *load effect* determined using the load combinations stipulated by the *applicable building code* including the *amplified seismic load*, shall satisfy both of the following requirements:

- (1) The *available strength* of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200 percent of the required strength.
- (2) The available strength for each flange splice shall be at least equal to $0.5 R_y F_y A_f$ (LRFD) or $(0.5/1.5) R_y F_y A_f$ (ASD), as appropriate, where $R_y F_y$ is the *expected yield stress* of the column material and A_f is the flange area of the smaller column connected.

Beveled transitions are not required when changes in thickness and width of flanges and webs occur in column splices where PJP groove welded joints are used.

Column web splices shall be either bolted or welded, or welded to one column and bolted to the other. In *moment frames* using bolted splices, plates or channels shall be used on both sides of the column web.

The centerline of column splices made with fillet welds or partial-joint-penetration groove welds shall be located 1.2 m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m, splices shall be at half the clear height.

Columns Not Part of the Seismic Load Resisting System

Splices of columns that are not a part of the SLRS shall satisfy the following:

- (1) Splices shall be located 1.2m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4m, splices shall be at half the clear height.
- (2) The *required shear strength* of column splices with respect to both orthogonal axes of the column shall be M_{pc}/H (LRFD) or $M_{pc}/1.5H$ (ASD), as appropriate, where M_{pc} is the lesser nominal plastic flexural strength of the column sections for the direction in question, and H is the story height.

10.20.8.4 Column Bases

The *required strength* of column bases shall be calculated in accordance with Sections 10.20.8.5.1, 10.20.8.5.2, and 10.20.8.5.3. The *available strength* of anchor rods shall be determined in accordance with *Specification* Section 10.10.3.

The available strength of concrete elements at the column base, including anchor rod embedment and reinforcing steel, shall be in accordance with Appendix D of Chapter 6 of Part 6 of this code.

Exception: The special requirements in Appendix D of Chapter 6 of Part 6 of this code, for “regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories” need not be applied.

Required Axial Strength

The required axial strength of column bases, including their attachment to the foundation, shall be the summation of the vertical components of the required strengths of the steel elements that are connected to the column base.

Required Shear Strength

The required shear strength of column bases, including their attachments to the foundations, shall be the summation of the horizontal component of the required strengths of the steel elements that are connected to the column base as follows:

- (1) For diagonal bracing, the horizontal component shall be determined from the required strength of bracing connections for the *seismic load resisting system (SLRS)*.
- (2) For columns, the horizontal component shall be at least equal to the lesser of the following:
 - (a) $2R_y F_y Z_x / H$ (LRFD) or $(2/1.5) R_y F_y Z_x / H$ (ASD), as appropriate, of the column

where

H = height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, mm.

(b) The shear calculated using the load combinations of the *applicable building code*, including the *amplified seismic load*.

Required Flexural Strength

The *required flexural strength* of column bases, including their attachment to the foundation, shall be the summation of the required strengths of the steel elements that are connected to the column base as follows:

- (1) For diagonal bracing, the required flexural strength shall be at least equal to the required strength of bracing connections for the SLRS.
- (2) For columns, the required flexural strength shall be at least equal to the lesser of the following:
 - (a) $1.1R_y F_y Z$ (LRFD) or $(1.1/1.5)R_y F_y Z$ (ASD), as appropriate, of the column or
 - (b) the moment calculated using the load combinations of the *applicable building code*, including the *amplified seismic load*.

10.20.8.5 H-Piles

Design of H-Piles

Design of H-piles shall comply with the provisions of the *Specification* regarding design of members subjected to combined loads. H-piles shall meet the requirements of Section 10.20.8.2.2.

Battered H-Piles

If battered (sloped) and vertical piles are used in a pile group, the vertical piles shall be designed to support the combined effects of the dead and live loads without the participation of the battered piles.

Tension in H-Piles

Tension in each pile shall be transferred to the pile cap by mechanical means such as shear keys, reinforcing bars or studs welded to the embedded portion of the pile. Directly below the bottom of the pile cap, each pile shall be free of attachments and welds for a length at least equal to the depth of the pile cross section.

10.20.9 Special Moment Frames (SMF)

10.20.9.1 Scope

Special moment frames (SMF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the *design earthquake*. SMF shall satisfy the requirements in this Section.

10.20.9.2 Beam-to-Column Connections

Requirements

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

- (1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.
- (2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.
- (3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E :

$$E = 2[1.1R_y M_p]/L_h \quad (10.20.9.1)$$

where

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y

M_p = nominal plastic flexural strength, (N-mm)

L_h = distance between plastic hinge locations, (mm)

When E as defined in Equation 10.20.9.1 is used in *ASD load combinations* that are additive with other transient loads and that are based on Chapter 2 (of Part 6), the 0.75 combination factor for transient loads shall not be applied to E .

Connections that accommodate the required interstory drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified above are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

Conformance Demonstration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 10.20.9.2.1 by one of the following:

- (a) Use of SMF connections designed in accordance with ANSI/AISC 358.
- (b) Use of a connection prequalified for SMF in accordance with Appendix 10A.
- (c) Provision of qualifying cyclic test results in accordance with Appendix 10.D.

Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

- (i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix 10.D.
- (ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix 10.D.

Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D, complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be *demand critical welds* as described in Section 10.20.7.3.2.

Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a *protected zone*, and shall meet the requirements of Section 10.20.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

10.20.9.3 Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

Shear Strength

The required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested or *prequalified connection*. As a minimum, the *required shear strength* of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength shall be $\phi_v R_v$ and the allowable shear strength shall be R_v/Ω_v where

$$\phi_v = 1.0 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$

and the *nominal shear strength*, R_v , according to the limit state of shear yielding, is determined as specified in *Specification* Section 10.10.10.6.

Panel Zone Thickness

The individual thicknesses, t , of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90 \quad (10.20.9.2)$$

where

t = thickness of column web or doubler plate, mm

d_z = panel zone depth between *continuity plates*, mm

w_z = panel zone width between column flanges, mm

Alternatively, when local buckling of the column web and doubler plate is prevented by using plug welds joining them, the total panel zone thickness shall satisfy Equation 10.20.9.2.

Panel Zone Doubler Plates

Doubler plates shall be welded to the column flanges using either a complete-joint-penetration groove-welded or fillet-welded joint that develops the available shear strength of the full doubler plate thickness. When doubler plates are placed against the column web, they shall be welded across the top and bottom edges to develop the proportion of the total force that is transmitted to the doubler plate. When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

Beam and Column Limitations

The requirements of Section 10.20.8.1 shall be satisfied, in addition to the following.

Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 10.20.8.2.2, unless otherwise qualified by tests.

Beam Flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or in a program of qualification testing in accordance with Appendix 10.D.

Continuity Plates

Continuity plates shall be consistent with the prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Column-Beam Moment Ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0 \quad (10.20.9.3)$$

$\sum M_{pc}^*$ = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\sum M_{pc}^*$ is determined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. It is permitted to take $\sum M_{pc}^* = \Sigma Z_c(F_{yc} - P_{uc}/A_g)$ (LRFD) or $\Sigma Z_c[(F_{yc}/1.5) - P_{uc}/A_g]$ (ASD), as appropriate. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

$\sum M_{pb}^*$ = the sum of the moments in the beams at the intersection of the beam and column centerlines. $\sum M_{pb}^*$ is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\sum M_{pb}^* = \Sigma(1.1R_yF_{yb}Z_b + M_{av})$ (LRFD) or $\Sigma[(1.1/1.5)R_yF_{yb}Z_b + M_{av}]$ (ASD), as appropriate. Alternatively, it is permitted to determine $\sum M_{pb}^*$ consistent with a prequalified connection design as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10.A, or in a program of qualification testing in accordance with Appendix 10.D. When connections with reduced beam sections are used, it is permitted to take $\sum M_{pb}^* = \Sigma(1.1R_yF_{yb}Z_{RBS} + M_{av})$ (LRFD) or $\Sigma[(1.1/1.5)R_yF_{yb}Z_{RBS} + M_{av}]$ (ASD), as appropriate.

A_g = gross area of column, mm

F_{yc} = specified minimum yield stress of column, MPa

M_{av} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on ASD load combinations, N-mm.

M_{uv} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on LRFD load combinations, N-mm

P_{ac} = required compressive strength using ASD load combinations, (positive number) N.

P_{uc} = required compressive strength using LRFD load combinations, (positive number) N

Z_b = plastic section modulus of the beam, mm³

Z_c = plastic section modulus of the column, mm³

Z_{RBS} = minimum plastic section modulus at the reduced beam section, mm³

Exception: This requirement does not apply if either of the following two conditions is satisfied:

(a) Columns with $P_{rc} < 0.3P_c$ for all load combinations other than those determined using the *amplified seismic load* that satisfy either of the following:

(i) Columns used in a one-story building or the top story of a multistory building.

(ii) Columns where: (1) the sum of the *available shear strengths* of all *exempted columns* in the story is less than 20 percent of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction; and (2) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33 percent of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

where

For design according to *Specification* Section 10.2.3.3 (LRFD),

$$P_c = F_{yc} A_g, \text{ N}$$

$P_{rc} = P_{uc}$, required compressive strength, using LRFD load combinations, N

For design according to *Specification* Section 10.2.3.4 (ASD),

$$P_c = F_{yc} A_g / 1.5, \text{ N}$$

$P_{rc} = P_{ac}$, required compressive strength, using ASD load combinations, N

(b) Columns in any story that has a ratio of available shear strength to *required shear strength* that is 50 percent greater than the story above.

10.20.9.4 Lateral Bracing at Beam-to-Column Connections

Braced Connections

Column flanges at beam-to-column connections require lateral bracing only at the level of the top flanges of the beams, when the webs of the beams and column are co-planar, and a column is shown to remain elastic outside of the panel zone. It shall be permitted to assume that the column remains elastic when the ratio calculated using Equation 10.20.9.3 is greater than 2.0.

When a column cannot be shown to remain elastic outside of the panel zone, the following requirements shall apply:

The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Lateral bracing shall be either direct or indirect.

Each column-flange lateral brace shall be designed for a *required strength* that is equal to 2 percent of the available beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf} / 1.5$ (ASD), as appropriate.

Unbraced Connections

A column containing a beam-to-column connection with no lateral bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral braces as the column height for buckling transverse to the seismic frame and shall conform to *Specification* Section 10.8, except that:

(1) The required column strength shall be determined from the appropriate load combinations, except that E shall be taken as the lesser of:

(a) The amplified seismic load.

(b) 125 percent of the frame *available strength* based upon either the beam available flexural strength or panel zone available shear strength.

(2) The slenderness L/r for the column shall not exceed 60.

The column *required flexural strength* transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Section 10.20.9.7.1.(2) in addition to the second-order moment due to the resulting column flange displacement.

Lateral Bracing of Beams

Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = 0.086r_y E/F_y$. Braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y ZF_y$ (LRFD) or $M_r = M_o = R_y ZF_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated forces, changes in cross-section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or in a program of qualification testing in accordance with Appendix 10.D.

The *required strength* of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u / h_o$ (LRFD) or $P_o = 0.06 M_o / h_o$ (ASD), as appropriate, where h_o is the distance between flange centroids; and the required stiffness shall meet the provisions of Equation 10.19.3.4 of Section 10.19.

Column Splices

Column splices shall comply with the requirements of Section 10.20.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Section 10.20.7.3.2. Weld tabs shall be removed. When column splices are not made with groove welds, they shall have a *required flexural strength* that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x / 1.5$ (ASD), as appropriate, of the smaller column. The required shear strength of column web splices shall be at least equal to $\Sigma M_{pc} / H$ (LRFD) or $\Sigma M_{pc} / 1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Exception: The *required strength* of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by inelastic analyses.

10.20.10 Intermediate Moment Frames (IMF)

10.20.10.1 Scope

Intermediate moment frames (IMF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the *design earthquake*. IMF shall meet the requirements in this Section.

10.20.10.2 Beam-to-Column Connections

Requirements

Beam-to-column connections used in the *seismic load resisting system (SLRS)* shall satisfy the requirements of Section 10.20.9.2.1, with the following exceptions:

- (1) The required *interstory drift angle* shall be a minimum of 0.02 radian.
- (2) The *required strength* in shear shall be determined as specified in Section 10.20.9.2.1, except that a lesser value of V_u or V_o , as appropriate, is permitted if justified by analysis. The *required shear strength* need not exceed the shear resulting from the application of appropriate *load combinations* using the *amplified seismic load*.

Conformance Demonstration

Conformance demonstration shall be as described in Section 10.20.9.2.2 to satisfy the requirements of Section 10.20.10.2.1 for IMF, except that a connection prequalified for IMF in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with

Appendix 10.D, complete joint penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be *demand critical welds* as described in Section 10.20.7.3.2.

Protected Zone

The region at each end of the beam subject to inelastic straining shall be treated as a *protected zone*, and shall meet the requirements of Section 10.20.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

No additional requirements beyond the *Specification*.

Beam and Column Limitations

The requirements of Section 10.20.8.1 shall be satisfied, in addition to the following.

Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 10.20.8.2.1, unless otherwise qualified by tests.

Beam Flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. Drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or in a program of qualification testing in accordance with Appendix 10.D.

Continuity Plates

Continuity plates shall be provided to be consistent with the prequalified connections designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Column-Beam Moment Ratio

No additional requirements beyond the *Specification*.

Lateral Bracing at Beam-to-Column Connections

No additional requirements beyond the *Specification*.

Lateral Bracing of Beams

Both flanges shall be laterally braced directly or indirectly. The unbraced length between lateral braces shall not exceed $0.17r_y E/F_y$. Braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_o = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam, and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated loads, changes in cross-section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. Where the design is based upon assemblies tested in accordance with Appendix 10.D, the placement of lateral bracing for the beams shall be consistent with that used in the tests or as required for prequalification in Appendix 10A. The *required strength* of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u / h_o$ (LRFD) or $P_o = 0.06 M_o / h_o$ (ASD), as appropriate, where h_o = distance between flange centroids; and the required stiffness shall meet the provisions of Equation 10.19.3.4 of Section 10.19.

Column Splices

Column splices shall comply with the requirements of Section 10.20.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Section 10.20.7.3.2.

10.20.11 Ordinary Moment Frames (OMF)

10.20.11.1 Scope

Ordinary moment frames (OMF) are expected to withstand minimal inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the *design earthquake*. OMF shall meet the requirements of this Section. Connections in conformance with Sections 10.20.9.2.1 and

10.20.9.5 or Sections 10.20.10.2.1 and 10.20.10.5 shall be permitted for use in OMF without meeting the requirements of Sections 10.20.11.2.1, 10.20.11.2.3, and 10.20.11.5

10.20.11.2 Beam-to-Column Connections

Beam-to-column connections shall be made with welds and/or high-strength bolts. Connections are permitted to be fully restrained (FR) or partially restrained (PR) moment connections as follows.

Requirements for FR Moment Connections

FR moment connections that are part of the *seismic load resisting system (SLRS)* shall be designed for a *required flexural strength* that is equal to $1.1R_y M_p$ (LRFD) or $(1.1/1.5)R_y M_p$ (ASD), as appropriate, of the beam or girder, or the maximum moment that can be developed by the system, whichever is less.

FR connections shall meet the following requirements.

(1) Where steel backing is used in connections with complete-joint-penetration (CJP) beam flange groove welds, steel backing and tabs shall be removed, except that top-flange backing attached to the column by a continuous fillet weld on the edge below the CJP groove weld need not be removed. Removal of steel backing and tabs shall be as follows:

(i) Following the removal of backing, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The reinforcing fillet shall have a minimum leg size of 8 mm.

(ii) Weld tab removal shall extend to within 3 mm of the base metal surface, except at *continuity plates* where removal to within 6 mm of the plate edge is acceptable. Edges of the weld tab shall be finished to a surface roughness value of 13 μm or better. Grinding to a flush condition is not required. Gouges and notches are not permitted. The transitional slope of any area where gouges and notches have been removed shall not exceed 1:5. Material removed by grinding that extends more than 2 mm below the surface of the base metal shall be filled with weld metal. The contour of the weld at the ends shall provide a smooth transition, free of notches and sharp corners.

(2) Where weld access holes are provided, they shall be as shown in Figure 10.20.11.1. The weld access hole shall have a surface roughness value not to exceed 13 μm , and shall be free of notches and gouges. Notches and gouges shall be repaired as required by the engineer of record. Weld access holes are prohibited in the beam web adjacent to the end-plate in bolted moment end-plate connections.

(3) The *required strength* of double-sided partial-joint-penetration groove welds and double-sided fillet welds that resist tensile forces in connections shall be $1.1R_y F_y A_g$ (LRFD) or $(1.1/1.5)R_y F_y A_g$ (ASD), as appropriate, of the connected element or part. Single-sided partial-joint-penetration groove welds and single-sided fillet welds shall not be used to resist tensile forces in the connections.

(4) For FR moment connections, the *required shear strength*, V_u or V_o , as appropriate, of the connection shall be determined using the following quantity for the earthquake load effect E :

$$E = 2[1.1R_y M_p]/L_h \quad (10.20.11.1)$$

Where this E is used in *ASD load combinations* that are additive with other transient loads and that are based on Chapter 2 (of Part 6), the 0.75 combination factor for transient loads shall not be applied to E .

Alternatively, a lesser value of V_u or V_o is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations in the *applicable building code* using the *amplified seismic load*

Requirements for PR Moment Connections

PR moment connections are permitted when the following requirements are met:

(1) Such connections shall be designed for the *required strength* as specified in Section 10.20.11.2.1 above.

(2) The *nominal flexural strength* of the connection, M_n , shall be no less than 50 percent of M_p of the connected beam or column, whichever is less.

(3) The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.

(4) For PR moment connections, V_u or V_o , as appropriate, shall be determined from the load combination above plus the shear resulting from the maximum end moment that the connection is capable of resisting.

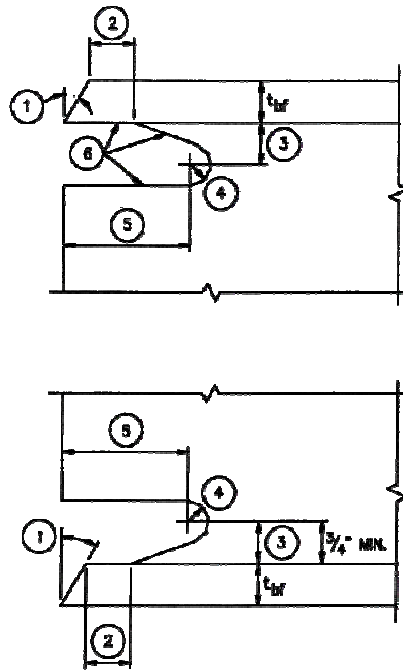


Fig. 10.20.11.1. Weld access hole detail (FEMA 350)

Notes: 1. Bevel as required for selected groove weld.

2. Larger of t_{bf} or 13 mm (plus $\frac{1}{2} t_{bf}$ or minus $\frac{1}{4} t_{bf}$)

3. $\frac{1}{4} t_{bf}$ to t_{bf} , 19 mm minimum (± 6 mm)

4. 10 mm minimum radius (plus not limited, minus 0)

5. $3 t_{bf}$ (± 13 mm)

Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25° .

Welds

Complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be *demand critical welds* as described in Section 10.20.7.3.2.

Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

No additional requirements beyond the *Specification*.

Beam and Column Limitations

No requirements beyond Section 10.20.8.1.

Continuity Plates

When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, *continuity plates* shall be provided in accordance with Section J10 of the *Specification*. Continuity plates shall also be required when:

$$t_{cf} < 0.54 (b_f t_{bf} F_{yb} / F_{yc})^{1/2}$$

or when,

$$t_{cf} < b_f / 6$$

Where continuity plates are required, the thickness of the plates shall be determined as follows:

For one-sided connections, continuity plate thickness shall be at least one half of the thickness of the beam flange.

For two-sided connections the continuity plates shall be at least equal in thickness to the thicker of the beam flanges.

The welded joints of the continuity plates to the column flanges shall be made with either complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds combined with reinforcing fillet welds, or two-sided fillet welds. The *required strength* of these joints shall not be less than the *available*

strength of the contact area of the plate with the column flange. The required strength of the welded joints of the continuity plates to the column web shall be the least of the following:

The sum of the available strengths at the connections of the continuity plate to the column flanges.

The *available shear strength* of the contact area of the plate with the column web.

The weld available strength that develops the available shear strength of the column panel zone.

The actual force transmitted by the stiffener.

Column-Beam Moment Ratio

No requirements.

Lateral Bracing at Beam-to-Column Connections

No additional requirements beyond the *Specification*.

Lateral Bracing of Beams

No additional requirements beyond the *Specification*.

Column Splices

Column splices shall comply with the requirements of Section 10.20.8.4.1.

10.20.12 Special Truss Moment Frames (STMF)

10.20.12.1 Scope

Special truss moment frames (STMF) are expected to withstand significant inelastic deformation within a specially designed segment of the truss when subjected to the forces from the motions of the *design earthquake*. STMF shall be limited to span lengths between columns not to exceed 20 m and overall depth not to exceed 1.8 m. The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. STMF shall meet the requirements in this Section.

10.20.12.2 Special Segment

Each horizontal truss that is part of the *seismic load resisting system (SLRS)* shall have a *special segment* that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

Panels within a special segment shall either be all Vierendeel panels or all X-braced panels; neither a combination thereof nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a *required strength* equal to 0.25 times the *nominal tensile strength* of the diagonal member. Bolted connections shall not be used for web members within the special segment. Diagonal web members within the special segment shall be made of flat bars of identical sections.

Splicing of chord members is not permitted within the special segment, nor within one-half the panel length from the ends of the special segment. The *required axial strength* of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03F_yA_g$ (LRFD) or $(0.03/1.5)F_yA_g$ (ASD), as appropriate.

The special segment shall be a *protected zone* meeting the requirements of Section 10.20.7.4.

Strength of Special Segment Members

The *available shear strength* of the *special segment* shall be calculated as the sum of the available shear strength of the chord members through flexure, and the shear strength corresponding to the *available tensile strength* and 0.3 times the *available compressive strength* of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25 percent of the *required vertical shear strength*. The *required axial strength* in the chord members, determined according to the limit state of tensile yielding, shall not exceed 0.45 times ϕP_n (LRFD) or P_n / Ω (ASD), as appropriate,

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where,

$$P_n = F_y A_g$$

The end connection of diagonal web members in the special segment shall have a *required strength* that is at least equal to the *expected yield strength*, in tension, of the web member, $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.

Strength of Non-Special Segment Members

Members and connections of STMF, except those in the *special segment* specified in Section 10.20.12.2, shall have a *required strength* based on the appropriate load combinations in the *applicable building code*, replacing the earthquake load term E with the lateral loads necessary to develop the *expected vertical shear strength* of the special segment V_{ne} (LRFD) or $V_{ne} / 1.5$ (ASD), as appropriate, at mid-length, given as:

$$V_{ne} = \frac{3.75 R_y M_{nc}}{L_s} + 0.075 EI \frac{L - L_s}{L_s^3} + R_y (P_{nt} + 0.3 P_{nc}) \sin \alpha \quad (10.20.12.1)$$

where

M_{nc} = nominal flexural strength of a chord member of the special segment, N-mm

EI = flexural elastic stiffness of a chord member of the special segment, N-mm²

L = span length of the truss, mm

L_s = length of the special segment, mm

P_{nt} = nominal tensile strength of a diagonal member of the special segment, N

P_{nc} = nominal compressive strength of a diagonal member of the special segment, N

α = angle of diagonal members with the horizontal

Width-Thickness Limitations

Chord members and diagonal web members within the special segment shall meet the requirements of Section 10.20.8.2.2.

Lateral Bracing

The top and bottom chords of the trusses shall be laterally braced at the ends of the *special segment*, and at intervals not to exceed L_p according to *Specification* Section 10.6 along the entire length of the truss. The *required strength* of each lateral brace at the ends of and within the special segment shall be

$$P_u = 0.06 R_y P_{nc} \text{ (LRFD) or}$$

$$P_a = (0.06/1.5) R_y P_{nc} \text{ (ASD), as appropriate,}$$

where P_{nc} is the *nominal compressive strength* of the special segment chord member. Lateral braces outside of the special segment shall have a required strength of

$$P_u = 0.02 R_y P_{nc} \text{ (LRFD) or}$$

$$P_a = (0.02/1.5) R_y P_{nc} \text{ (ASD), as appropriate.}$$

The required brace stiffness shall meet the provisions of Equation 10.19.2.4 of Section 10.19, where

$$P_r = P_u = R_y P_{nc} \text{ (LRFD) or}$$

$$P_r = P_a = R_y P_{nc} / 1.5 \text{ (ASD), as appropriate.}$$

10.20.13 Special Concentrically Braced Frames (SCBF)

10.20.13.1 Scope

Special concentrically braced frames (SCBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the *design earthquake*. SCBF shall meet the requirements in this Section.

10.20.13.2 Members Slenderness

Bracing members shall have $Kl/r \leq 4\sqrt{E/F_y}$.

Exception: Braces with $4\sqrt{E/F_y} < Kl/r \leq 200$ are permitted in frames in which the available strength of the column is at least equal to the maximum load transferred to the column considering R_y (LRFD) or $(1/1.5)R_y$ (ASD), as appropriate, times the nominal strengths of the connecting brace elements of the building. Column

forces need not exceed those determined by inelastic analysis, nor the maximum load effects that can be developed by the system.

Required Strength

Where the effective net area of bracing members is less than the gross area, the *required tensile strength* of the brace based upon the limit state of fracture in the net section shall be greater than the lesser of the following:

- (a) The *expected yield strength*, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.
- (b) The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

Lateral Force Distribution

Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the *required strength* resulting from the application of the appropriate load combinations stipulated by the *applicable building code* including the *amplified seismic load*. For the purposes of this provision, a line of bracing is defined as a single line or parallel lines with a plan offset of 10 percent or less of the building dimension perpendicular to the line of bracing.

Width-Thickness Limitations

Column and brace members shall meet the requirements of Section 10.20.8.2.2.

Built-up Members

The spacing of stitches shall be such that the slenderness ratio l/r of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member.

The sum of the *available shear strengths* of the stitches shall equal or exceed the available tensile strength of each element. The spacing of stitches shall be uniform. Not less than two stitches shall be used in a built-up member. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

Exception: Where the buckling of braces about their critical buckling axis does not cause shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio l/r of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

Required Strength of Bracing Connections

Required Tensile Strength

The *required tensile strength* of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:

- (a) The *expected yield strength*, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.
- (b) The maximum *load effect*, indicated by analysis that can be transferred to the brace by the system.

Required Flexural Strength

The *required flexural strength* of bracing connections shall be equal to $1.1R_y M_p$ (LRFD) or $(1.1/1.5)R_y M_p$ (ASD), as appropriate, of the brace about the critical buckling axis.

Exception: Brace connections that meet the requirements of Section 10.20.13.3.1 and can accommodate the inelastic rotations associated with brace post-buckling deformations need not meet this requirement.

Required Compressive Strength

Bracing connections shall be designed for a *required compressive strength* based on buckling limit states that is at least equal to $1.1R_y P_n$ (LRFD) or $(1.1/1.5)R_y P_n$ (ASD), as appropriate, where P_n is the *nominal compressive strength* of the brace.

Special Bracing Configuration Requirements

V-Type and Inverted-V-Type Bracing

V-type and inverted V-type SCBF shall meet the following requirements:

- (1) The *required strength* of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the *applicable building code* assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the beam shall be determined as follows:

- (a) The forces in all braces in tension shall be assumed to be equal to $R_y F_y A_g$.
- (b) The forces in all adjoining braces in compression shall be assumed to be equal to $0.3P_n$.

(2) Beams shall be continuous between columns. Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equation 10.15.7.2 and 10.15.7.3 of Section 10.15. Lateral braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_u = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

K-Type Bracing

K-type braced frames are not permitted for SCBF.

Column Splices

In addition to meeting the requirements in Section 10.20.8.4, column splices in SCBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members. The required shear strength shall be $\Sigma M_{pc} / H$ (LRFD) or $\Sigma M_{pc} / 1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Protected Zone

The *protected zone* of bracing members in SCBF shall include the center one-quarter of the brace length, and a zone adjacent to each connection equal to the brace depth in the plane of buckling. The protected zone of SCBF shall include elements that connect braces to beams and columns and shall satisfy the requirements of Section 10.20.7.4.

10.20.14 Ordinary Concentrically Braced Frames (OCBF)

10.20.14.1 Scope

Ordinary concentrically braced frames (OCBF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the *design earthquake*. OCBF shall meet the requirements in this Section. OCBF above the isolation system in *seismically isolated structures* shall meet the requirements of Sections 10.20.14.4 and 10.20.14.5 and need not meet the requirements of Sections 10.20.14.2 and 10.20.14.3.

10.20.14.2 Bracing Members

Bracing members shall meet the requirements of Section 10.20.8.2.2.

Exception: HSS braces that are filled with concrete need not comply with this provision.

Bracing members in K, V, or inverted-V configurations shall have $KL/r \leq 4\sqrt{E/F_y}$.

Special Bracing Configuration Requirements

Beams in V-type and inverted V-type OCBF and columns in K-type OCBF shall be continuous at bracing connections away from the beam-column connection and shall meet the following requirements:

(1) The *required strength* shall be determined based on the load combinations of the *applicable building code* assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the member shall be determined as follows:

- (a) The forces in braces in tension shall be assumed to be equal to $R_y F_y A_g$. For V-type and inverted V-type OCBF, the forces in braces in tension need not exceed the maximum force that can be developed by the system.
- (b) The forces in braces in compression shall be assumed to be equal to $0.3P_n$.

(2) Both flanges shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equations 10.15.7.2 and 10.15.7.3 of Section 10.15. Lateral braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_u = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the bracing, unless the member has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

Bracing Connections

The *required strength* of bracing connections shall be determined as follows.

(1) For the limit state of bolt slip, the required strength of bracing connections shall be that determined using the load combinations stipulated by the *applicable building code*, not including the *amplified seismic load*.

(2) For other limit states, the required strength of bracing connections is the *expected yield strength*, in tension, of the brace, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.

Exception: The required strength of the brace connection need not exceed either of the following:

- (a) The maximum force that can be developed by the system
- (b) A load effect based upon using the amplified seismic load

OCBF above Seismic Isolation Systems

Bracing Members

Bracing members shall meet the requirements of Section 10.20.8.2.2 and shall have

$$KL/r \leq 4\sqrt{E/F_y}.$$

K-Type Bracing

K-type braced frames are not permitted.

V-Type and Inverted-V-Type Bracing

Beams in V-type and inverted V-type bracing shall be continuous between columns.

10.20.15 Eccentrically Braced Frames (EBF)

10.20.15.1 Scope

Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the *links* when subjected to the forces resulting from the motions of the *design earthquake*. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened links, except where permitted in this Section. In buildings exceeding five stories in height, the upper story of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the *applicable building code*. EBF shall meet the requirements in this Section.

10.20.15.2 Links

Limitations

Links shall meet the requirements of Section 10.20.8.2.2.

The web of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

Shear Strength

Except as limited below, the link design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n / Ω_v , according to the limit state of shear yielding shall be determined as follows:

V_n = nominal shear strength of the link, equal to the lesser of V_p or $2M_p / e$, N

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

where

$$M_p = F_y Z, \text{ N-mm}$$

$$V_p = 0.6 F_y A_{wv}, \text{ N}$$

e = link length, mm

$$A_{wv} = (d - 2t_f)t_w$$

The effect of axial force on the link available shear strength need not be considered if

$$P_u \leq 0.15 P_y \text{ (LRFD) or } P_a \leq (0.15/1.5) P_y \text{ (ASD), as appropriate.}$$

Where,

P_u = required axial strength using LRFD load combinations, N

P_a = required axial strength using ASD load combinations, N

P_y = nominal axial yield strength = $F_y A_g$, N

If $P_u > 0.15P_y$ (LRFD) or $P_a > (0.15/1.5)P_y$ (ASD), as appropriate, the following additional requirements shall be met:

(1) The available shear strength of the link shall be the lesser of

$\phi_v V_{pa}$ and $2\phi_v M_{pa}/e$ (LRFD)

or

V_{pa}/Ω_v and $2(M_{pa}/e)/\Omega_v$ (ASD), as appropriate, where

$\phi_v = 0.90$ (LRFD), $\Omega_v = 1.67$ (ASD)

$$V_{pa} = V_p \sqrt{1 - (P_r / P_c)^2} \quad (10.20.15.1)$$

$$M_{pa} = 1.18 M_p [1 - (P_r / P_c)] \quad (10.20.15.2)$$

$P_r = P_u$ (LRFD) or P_a (ASD), as appropriate

$P_c = P_y$ (LRFD) or $P_y/1.5$ (ASD), as appropriate

(2) The length of the link shall not exceed:

$$(a) [1.15 - 0.5\rho'(A_w/A_g)]1.6M_p/V_p \text{ when } \rho'(A_w/A_g) \geq 0.3 \quad (10.20.15.3)$$

Nor,

$$(b) 1.6 M_p/V_p \text{ when } \rho'(A_w/A_g) < 0.3 \quad (10.20.15.4)$$

Where,

$$A_w = (d - 2t_f)t_w$$

$$\rho' = P_r / V_r$$

and where,

$V_r = V_u$ (LRFD) or V_a (ASD), as appropriate

V_u = required shear strength based on LRFD load combinations.

V_a = required shear strength based on ASD load combinations.

Link Rotation Angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift, Δ . The link rotation angle shall not exceed the following values:

(a) 0.08 radians for links of length $1.6M_p/V_p$ or less.

(b) 0.02 radians for links of length $2.6M_p/V_p$ or greater.

(c) The value determined by linear interpolation between the above values for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

Link Stiffeners

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ or 10 mm, whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

(a) Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.08 radian or $(52t_w - d/5)$ for link rotation angles of 0.02 radian or less. Linear interpolation shall be used for values between 0.08 and 0.02 radian.

(b) Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.

(c) Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) above.

- (d) Intermediate web stiffeners are not required in links of lengths greater than $5M_p/V_p$.
- (e) Intermediate web stiffeners shall be full depth. For links that are less than 635 mm in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 10 mm, whichever is larger, and the width shall be not less than $(b_f/2) t_w$. For links that are 635 mm in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The *required strength* of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y/1.5$ (ASD), as appropriate, where A_{st} is the area of the stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $A_{st}F_y/4$ (LRFD) or $A_{st}F_y/4(1.5)$ (ASD).

Link-to-Column Connections

Link-to-column connections must be capable of sustaining the maximum *link rotation angle* based on the length of the link, as specified in Section 10.20.15.2.3. The strength of the connection measured at the column face shall equal at least the nominal shear strength of the link, V_n , as specified in Section 10.20.15.2.2 at the maximum link rotation angle.

Link-to-column connections shall satisfy the above requirements by one of the following:

- (a) Use a connection *prequalified* for EBF in accordance with Appendix 10A.
- (b) Provide qualifying cyclic test results in accordance with Appendix 10.D. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
 - (i) Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Appendix 10.D.
 - (ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix 10.D.

Exception: Where reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length, the link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such links are used and the link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the *available strength* of the reinforced section and the connection equals or exceeds the *required strength* calculated based upon the strain-hardened link as described in Section 10.20.15.6. Full depth stiffeners as required in Section 10.20.15.3 shall be placed at the link-to-reinforcement interface.

Lateral Bracing of Link

Lateral bracing shall be provided at both the top and bottom *link* flanges at the ends of the link. The *required strength* of each lateral brace at the ends of the link shall be $P_b = 0.06 M_r / h_o$, where h_o is the distance between flange centroids in mm.

For design according to *Specification* Section 10.2.3.3 (LRFD)

$$M_r = M_{u,exp} = R_y Z F_y$$

For design according to *Specification* Section 10.2.3.4 (ASD)

$$M_r = M_{u,exp} / 1.5$$

The required brace stiffness shall meet the provisions of Equation 10.19.3.4 of Section 10.19, where M_r is defined above, $C_d = 1$, and L_b is the link length.

10.20.15.3 Diagonal Brace and Beam Outside of Link

Diagonal Brace

The *required combined axial and flexural strength* of the diagonal brace shall be determined based on load combinations stipulated by the *applicable building code*. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E , where Q_1 is defined as the axial forces and moments generated by at least 1.25 times the *expected nominal shear strength* of the link $R_y V_n$, where V_n is as defined in Section 10.20.15.2.2. The *available strength* of the diagonal brace shall comply with *Specification* Section 10.10. Brace members shall meet the requirements of Section 10.20.8.2.1.

Beam Outside Link

The required combined axial and flexural strength of the beam outside of the link shall be determined based on load combinations stipulated by the applicable building code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E where Q_1 is defined as the forces generated by at least 1.1 times the expected nominal shear strength of the link, $R_y V_n$, where V_n is as defined in Section 10.20.15.2.2. The available strength of the beam outside of the link shall be determined by the *Specification*, multiplied by R_y .

At the connection between the diagonal brace and the beam at the link end of the brace, the intersection of the brace and beam centerlines shall be at the end of the link or in the link.

Bracing Connections

The *required strength* of the diagonal brace connections, at both ends of the brace, shall be at least equal to the required strength of the diagonal brace, as defined in Section 10.20.15.6.1. The diagonal brace connections shall also satisfy the requirements of Section 10.20.13.3.3.

No part of the diagonal brace connection at the link end of the brace shall extend over the link length. If the brace is designed to resist a portion of the link end moment, then the diagonal brace connection at the link end of the brace shall be designed as a fully-restrained moment connection.

Beam-to-Column Connections

If the EBF system factors in the *applicable building code* require moment resisting connections away from the *link*, then the beam-to-column connections away from the link shall meet the requirements for beam-to-column connections for OMF specified in Sections 10.20.11.2 and 10.20.11.5.

If the EBF system factors in the applicable building code do not require moment resisting connections away from the link, then the beam-to-column connections away from the link are permitted to be designed as pinned in the plane of the web.

Required Strength of Columns

In addition to the requirements in Section 10.20.8.3, the *required strength* of columns shall be determined from load combinations as stipulated by the *applicable building code*, except that the seismic load E shall be the forces generated by 1.1 times the *expected nominal shear strength* of all links above the level under consideration. The expected nominal shear strength of a link is $R_y V_n$, where V_n is as defined in Section 10.20.15.2.2. Column members shall meet the requirements of Section 10.20.8.2.2.

Protected Zone

Links in EBFs are a *protected zone*, and shall satisfy the requirements of Section 10.20.7.4. Welding on links is permitted for attachment of link stiffeners, as required in Section 10.20.15.3.

Demand Critical Welds

Complete-joint-penetration groove welds attaching the *link* flanges and the link web to the column are *demand critical welds*, and shall satisfy the requirements of Section 10.20.7.3.2.

10.20.16 Buckling-Restrained Braced Frames (BRBF)

10.20.16.1 Scope

Buckling-restrained braced frames (BRBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the *design earthquake*. BRBF shall meet the requirements in this Section. Where the *applicable building code* does not contain design coefficients for BRBF, the provisions of Appendix 10.C shall apply.

10.20.16.2 Bracing Members

Bracing members shall be composed of a structural steel core and a system that restrains the steel core from buckling.

Steel Core

The *steel core* shall be designed to resist the entire axial force in the brace.

The brace *design axial strength*, ϕP_{ysc} (LRFD), and the brace *allowable axial strength*, P_{ysc}/Ω (ASD), in tension and compression, according to the limit state of yielding, shall be determined as follows:

$$P_{ysc} = F_{ysc} A_{sc} \quad (10.20.16.1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where

F_{ysc} = specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa

A = net area of steel core, mm²

Plates used in the steel core that are 50 mm thick or greater shall satisfy the minimum notch toughness requirements of Section 10.20.6.3.

Splices in the steel core are not permitted.

Buckling-Restraining System

The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the steel core for deformations corresponding to 2.0 times the *design story drift*. The buckling-restraining system shall not be permitted to buckle within deformations corresponding to 2.0 times the design story drift.

Testing

The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Appendix 10.E. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace sub-assembly that includes brace connection rotational demands complying with Appendix 10.E, Section 10.E.4 and the other shall be either a uniaxial or a sub-assembly test complying with Appendix 10.E, Section 10.E.5. Both test types are permitted to be based upon one of the following:

- (a) Tests reported in research or documented tests performed for other projects.
- (b) Tests that are conducted specifically for the project.

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests shall be permitted to qualify a design when the provisions of Appendix 10.E are met.

Adjusted Brace Strength

Where required by these Provisions, bracing connections and adjoining members shall be designed to resist forces calculated based on the *adjusted brace strength*.

The adjusted brace strength in compression shall be $\beta \omega R_y P_{ysc}$. The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$.

Exception: The factor R_y need not be applied if P_{ysc} is established using yield stress determined from a coupon test.

The compression strength adjustment factor, β , shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Appendix 10.E, Section 10.E.6.3 for the range of deformations corresponding to 2.0 times the design story drift. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0. The strain hardening adjustment factor, ω , shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Appendix 10.E, Section 10.E.6.3 (for the range of deformations corresponding to 2.0 times the design story drift) to F_{ysc} of the test specimen. The larger value of ω from the two required qualification tests shall be used. Where the tested steel core material does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

10.20.16.3 Bracing Connections

Required Strength

The *required strength* of bracing connections in tension and compression (including beam-to-column connections if part of the bracing system) shall be 1.1 times the *adjusted brace strength* in compression (LRFD) or 1.1/1.5 times the adjusted brace strength in compression (ASD).

Gusset Plates

The design of connections shall include considerations of local and overall buckling. Bracing consistent with that used in the tests upon which the design is based is required.

Special Requirements Related to Bracing Configuration

V-type and inverted-V-type braced frames shall meet the following requirements:

(1) The *required strength* of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the *applicable building code* assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the vertical and horizontal earthquake effect, E , on the beam shall be determined from the *adjusted brace strengths* in tension and compression.

(2) Beams shall be continuous between columns. Both flanges of beams shall be laterally braced. Lateral braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_o = R_y Z F_y$ (LRFD) or $M_r = M_o = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

For purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam under the loading defined in Section 10.20.16.4(1).

K-type braced frames are not permitted for BRBF.

Beams and Columns

Beams and columns in BRBF shall meet the following requirements.

Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 10.20.8.2.2.

Required Strength

The *required strength* of beams and columns in BRBF shall be determined from load combinations as stipulated in the *applicable building code*. For load combinations that include earthquake effects, the earthquake effect, E , shall be determined from the *adjusted brace strengths* in tension and compression.

The required strength of beams and columns need not exceed the maximum force that can be developed by the system.

Splices

In addition to meeting the requirements in Section 10.20.8.4, column splices in BRBF shall be designed to develop 50 percent of the lesser *available flexural strength* of the connected members, determined based on the limit state of yielding. The *required shear strength* shall be $\Sigma M_{pc} / H$ (LRFD) or $\Sigma M_{pc} / 1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Protected Zone

The *protected zone* shall include the steel core of bracing members and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Section 10.20.7.4.

10.20.17 Special Plate Shear Walls (SPSW)

10.20.17.1 Scope

Special plate shear walls (SPSW) are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the design earthquake. The horizontal boundary elements (HBEs) and vertical boundary elements (VBEs) adjacent to the webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded webs, except that plastic hinging at the ends of HBEs is permitted. SPSW shall meet the requirements of this Section. Where the applicable building code does not contain design coefficients for SPSW, the provisions of Appendix 10.C shall apply.

10.20.17.2 Webs

Shear Strength

The panel design shear strength, ϕV_n (LRFD), and the allowable shear strength, V_n/Ω (ASD), according to the limit state of shear yielding, shall be determined as follows:

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha \quad (10.20.17.1)$$

$$\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

where

t_w = thickness of the web, mm.

L_{cf} = clear distance between VBE flanges, mm

α is the angle of web yielding in radians, as measured relative to the vertical, and it is given by:

$$\tan^4 \alpha = \frac{1 + \frac{t_w L}{2A_c}}{1 + t_w h \left(\frac{1}{A_b} + \frac{h^3}{360I_c L} \right)} \quad (10.20.17.2)$$

h = distance between HBE centerlines, mm

A_b = cross-sectional area of a HBE, mm²

A_c = cross-sectional area of a VBE, mm²

I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, mm⁴

L = distance between VBE centerlines, mm

Panel Aspect Ratio

The ratio of panel length to height, L/h , shall be limited to $0.8 < L/h \leq 2.5$.

Openings in Webs

Openings in webs shall be bounded on all sides by HBE and VBE extending the full width and height of the panel, respectively, unless otherwise justified by testing and analysis.

Connections of Webs to Boundary Elements

The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle α , defined by Equation 10.20.17.2.

10.20.17.3 Horizontal and Vertical Boundary Elements

Required Strength

In addition to the requirements of Section 10.20.8.3, the *required strength* of VBE shall be based upon the forces corresponding to the *expected yield strength*, in tension, of the web calculated at an angle α .

The required strength of HBE shall be the greater of the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α or that determined from the load combinations in the *applicable building code* assuming the web provides no support for gravity loads.

The beam-column moment ratio provisions in Section 10.20.9.6 shall be met for all HBE/VBE intersections without consideration of the effects of the webs.

HBE-to-VBE Connections

HBE-to-VBE connections shall satisfy the requirements of Section 10.20.11.2. The required shear strength, V_n , of a HBE-to-VBE connection shall be determined in accordance with the provisions of Section 10.20.11.2, except that the required shear strength shall not be less than the shear corresponding to moments at each end equal to $1.1R_y M_p$ (LRFD) or $(1.1/1.5)R_y M_p$ (ASD), as appropriate, together with the shear resulting from the *expected yield strength* in tension of the webs yielding at an angle α .

Width-Thickness Limitations

HBE and VBE members shall meet the requirements of Section 10.20.8.2.2.

Lateral Bracing

HBE shall be laterally braced at all intersections with VBE and at a spacing not to exceed $0.086r_y E/F_y$. Both flanges of HBE shall be braced either directly or indirectly. The required strength of lateral bracing shall be at least 2 percent of the HBE flange *nominal strength*, $F_y b_f t_f$. The required stiffness of all lateral bracing shall be determined in accordance with Equation 10.19.3.4 of Section 10.19. In these equations, M_r shall be computed as $R_y ZF_y$ (LRFD) or M_r shall be computed as $R_y ZF_y/1.5$ (ASD), as appropriate, and $C_d = 1.0$.

VBE Splices

VBE splices shall comply with the requirements of Section 10.20.8.4.

Panel Zones

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section 10.20.9.3.

Stiffness of Vertical Boundary Elements

The VBE shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c , not less than $0.00307t_w h^4/L$.

10.20.18 Quality Assurance Plan**Scope**

When required by the *applicable building code* or the engineer of record, a *quality assurance plan* shall be provided. The quality assurance plan shall include the requirements of Appendix 10.B.

Chapter 11

TIMBER

11.1 SCOPE

- 11.1.1** This Section relates to the use of structural timber in structures or elements of structures connected together by fasteners/fastening techniques.
- 11.1.2** This shall not be interpreted to prevent the use of material or methods of design or construction not specifically mentioned herein; and the methods of design may be based on analytical and engineering principles, or reliable test data, or both, that demonstrate the safety and serviceability of the resulting structure. Nor is the classification of timber into strength groups to be interpreted as preventing the use of design data desired for a particular timber or grade of timber on the basis of reliable tests.

11.2 TERMINOLOGY

- 11.2.1** This section provides an alphabetical list of the terms used in this chapter of the Code. In case of any conflict or contradiction between a definition given in this section and that in Part 1, the meaning provided in this section shall govern for interpretation of the provisions of this chapter.

11.2.2 Structural Purpose Definitions

11.2.2.1 Beam, Built-Up-Laminated

A beam made by joining layers of timber together with mechanical fastenings, so that the grain of all layers is essentially parallel.

11.2.2.2 Beam, Glued-Laminated

A beam made by bonding layers of veneers or timber with an adhesive, so that grain of all laminations is essentially parallel.

11.2.2.3 Diaphragm, Structural

A structural element of large extent placed in a building as a wall, or roof, and made use of to resist horizontal forces such as wind or earthquakes-acting parallel to its own plane.

11.2.2.4 Duration of Load

Period during which a member or a complete structure is stressed as a consequence of the loads applied.

11.2.2.5 Edge Distance

The distance measured perpendicular to grain from the centre of the connector to the edge of the member.

11.2.2.6 End Distance

The distance measured parallel to grain of the member from the centre of the connector to the closest end of timber.

11.2.2.7 Finger Joint

Joint produced by connecting timber members end-to-end by cutting profiles (tapered projections) in the form of V-shaped grooves to the ends of timber planks or scantlings to be joined, gluing the interfaces and then mating the two ends together under pressure.

11.2.2.8 **Fundamental or Ultimate Stress**

The stress which is determined on small clear specimen of timber, in accordance with good practice; and does not take into account the effect of naturally occurring characteristics and other factors.

11.2.2.9 **Inside Location**

Position in buildings in which timber remains continuously dry or protected from weather.

11.2.2.10 **Laminated Veneer Lumber**

A structural composite made by laminating veneers, 1.5 mm to 4.2 mm thick, with suitable adhesive and with the grain of veneers in successive layers aligned along the longitudinal (length) dimension of the composite.

11.2.2.11 **Loaded Edge Distance**

The distance measured from the centre to the edge towards which the load induced by the connector acts, and the unloaded edge distance is the one opposite to the loaded edge.

11.2.2.12 **Location**

A term generally referred to as exact place where a timber is used in building.

11.2.2.13 **Outside Location**

Position in buildings in which timbers are occasionally subjected to wetting and drying as in the case of open sheds and outdoor exposed structures.

11.2.2.14 **Permissible Stress**

Stress obtained by applying factor of safety to the ultimate stress.

11.2.2.15 **Sandwich, Structural**

A layered construction comprising a combination or relatively high-strength facing material intimately bonded to and acting integrally with a low density core material.

11.2.2.16 **Spaced Column**

Two column sections adequately connected together by glue, bolts, screws or otherwise.

11.2.2.17 **Structure, Permanent**

Structural units in timber which are constructed for a long duration and wherein adequate protection and design measures have initially been incorporated to render the structure serviceable for the required life.

11.2.2.18 **Structure, Temporary**

Structures which are erected for a short period, such as hutments at project sites, for rehabilitation, temporary defence constructions, exhibition structures, etc.

11.2.2.19 **Structural Element**

The component timber members and joints which make up a resulting structural assembly.

11.2.2.20 **Structural Grades**

Grades defining the maximum size of strength reducing natural characteristics (knots, sloping grain, etc) deemed permissible in any piece of structural timber within designated structural grade classification.

11.2.2.21 **Structural Timber**

Timber in which strength is related to the anticipated in-service use as a controlling factor in grading and selection and/or stiffness.

11.2.2.22 **Termite**

An insect of the order Isopteran which may burrow in the wood or wood products of a building for food or shelter.

11.2.2.23 Wet Location

Position in buildings in which timbers are almost continuously damp or wet in contact with the earth or water, such as piles and timber foundations.

11.2.3 Definitions of Defects in Timber

11.2.3.1 Check

A separation of fibres extending along the grain which is confined to one face of a piece of wood.

11.2.3.2 Compression Wood

Abnormal wood which is formed on the lower sides of branches and inclined stems of coniferous trees. It is darker and harder than normal wood but relatively low in strength for its weight. It can be usually identified by wide eccentric growth rings with abnormally high proportion of growth latewood.

11.2.3.3 Dead Knot

A knot in which the layers of annual growth are not completely intergrown with those of the adjacent wood. It is surrounded by pitch or bark. The encasement may be partial or complete.

11.2.3.4 Decay or Rot

Disintegration of wood tissue caused by fungi (wood destroying) or other microorganisms.

11.2.3.5 Decayed Knot

A knot softer than the surrounding wood and containing decay.

11.2.3.6 Diameter of Knot

The maximum distance between the two points farthest apart on the periphery of a round knot, on the face on which it becomes visible. In the case of a spike or a splay knot, the maximum width of the knot visible on the face on which it appears shall be taken as its diameter.

11.2.3.7 Discoloration

A change from the normal colour of the wood which does not impair the strength of the wood.

11.2.3.8 Knot

A branch base or limb embedded in the tree or timber by natural growth.

11.2.3.9 Knot Hole

A hole left as a result of the removal of a knot.

11.2.3.10 Live Knot

A knot free from decay and other defects, in which the fibres are firmly intergrown with those of the surrounding wood. Syn. 'Integrown knot'; *cf.* 'Dead Knot'.

11.2.3.11 Loose Grain (Loosened Grain)

A defect on a flat sawn surface caused by the separation or raising of wood fibres along the growth rings; C\$ 'Raised Grain'.

11.2.3.12 Loose Knot

A knot that is not held firmly in place by growth or position, and that cannot be relied upon to remain in place; *cf.* 'Tight Knot'.

11.2.3.13 Mould

A soft vegetative growth that forms on wood in damp, stagnant atmosphere. It is the least harmful type of fungus, usually confined to the surface of the wood.

11.2.3.14 **Pitch Pocket**

Accumulation of resin between growth rings of coniferous wood as seen on the cross section.

11.2.3.15 **Sap Stain**

Discoloration of the sapwood mainly due to fungi.

11.2.3.16 **Sapwood**

The outer layer of log, which in the growing tree contain living cells and food material. The sapwood is usually lighter in colour and is readily attacked by insects and fungi.

11.2.3.17 **Shake**

A partial or complete separation between adjoining layers of tissues as seen in end surfaces.

11.2.3.18 **Slope of Grain**

The inclination of the fibres to the longitudinal axis of the member.

11.2.3.19 **Sound Knot**

A tight knot free from decay, which is solid across its face, and at least as hard as the surrounding wood.

11.2.3.20 **Split**

A crack extending from one face of a piece of wood to another and mns along the grain of the piece.

11.2.3.21 **Tight Knot**

A knot so held by growth or position as to remain firm in position in the piece of wood; C\$ 'Loose Knot'.

11.2.3.22 **Wane**

The original rounded surface of a tree remaining on a piece of converted timber.

11.2.3.23 **Warp**

A deviation in sawn timber from a true plane surface or distortion due to stresses causing departure from a true plane.

11.2.3.24 **Warm Holes**

Cavities caused by worms.

11.3 **SYMBOLS**

11.3.1 For the purpose of this Section, the following letter symbols shall have the meaning indicated against each:

a = Projected area of bolt in main member ($t' \times d_3$), mm²

B = Width of the beam, mm

C = Concentrated load, N

D = Depth of beam, mm

D_1 = Depth of beam at the notch, mm

D_2 = Depth of notch, mm

d = Dimension of least side of column, mm

d_1 = Least overall width of box column, mm

d_2 = Least overall dimension of core in box column, mm

d_3 = Diameter of bolt, mm

d_f = Bolt-diameter factor

e = Length of the notch measured along the beam span from the inner edge of the support to the farthest edge of the notch, mm

E = Modulus of elasticity in bending, N/mm^2

F = Load acting on a bolt at an angle to grain, N

f_{ab} = Calculated bending stress in extreme fibre, N/mm^2

f_{ac} = Calculated average axial compressive stress, N/mm^2

f_{at} = Calculated axial tensile stress, N/mm^2

f_b = Permissible bending stress on the extreme fibre, N/mm^2

f_c = Permissible stress in axial compression, N/mm^2

f_{cn} = Permissible stress in compression normal (perpendicular) to grain, N/mm^2

f_{cp} = Permissible stress in compression parallel to grain, N/mm^2

f_c^0 = Permissible compressive stress in the direction of the line of action of the load, N/mm^2

f_t = Permissible stress in tension parallel to grain, N/mm^2

H = Horizontal shear stress, N/mm^2

I = Moment of inertia of a section, mm^4

K = Coefficient in deflection depending upon type and criticality of loading on beam

K_1 = Modification factor for change in slope of grain

K_2 = Modification factor for change in duration of loadings

$\left. \begin{array}{l} K_3, \\ K_4, \\ K_5, \\ \text{and} \\ K_6 \end{array} \right\} = \text{Form factors}$

K_7 = Modification factor for bearing stress

K_8 = Constant equal to $0.584 \sqrt{\frac{E}{f_{cp}}}$

K_9 = Constant equal to $\frac{\pi}{2} \sqrt{\frac{UE}{5qf_{cp}}}$

K_{10} = Constant equal to $0.584 \sqrt{\frac{2.5E}{f_{cp}}}$

L = Span of a beam or truss, mm

M = Maximum bending moment in beam N/mm^2

N = Total number of bolts in the joint

n = Shank diameter of the nail, mm

P = Load on bolt parallel to grain, N

P_1 = Ratio of the thickness of the compression flange to the depth of the beam

Q = Statical moment of area above or below the neutral axis about neutral axis, mm^3

q = Constant for particular thickness of plank

q_1 = Ratio of the total thickness of web or webs to the overall width of the beam

R = Load on bolt perpendicular (normal) to grain, N

S = Unsupported overall length of column, mm

t = Nominal thickness of planks used in forming box type column, mm

t' = Thickness of main member, mm

U = Constant for a particular thickness of the plank

V = Vertical end reaction or shear at a section, N

W = Total uniform load, N

x = Distance from reaction to load, mm

γ = A factor determining the value of form factor K_4

δ = Deflection at middle of beam, mm

θ = Angle of load to grain direction

Z = Section modulus of beam, mm³

λ_1 = Percentage factor for t'/d_3 ratio, parallel to grain

λ_2 = Percentage factor for t'/d_3 ratio, perpendicular to grain

11.4 MATERIALS

11.4.1 Species of Timber

The species of timber recommended for structural purposes are given in Table 11.4.1.

11.4.1.1 Grouping

Species of timber recommended for constructional purposes are classified in three groups on the basis of their strength properties, namely, modulus of elasticity (E) and extreme fibre stress in bending and tension (f_b).

The characteristics of these groups are given below:

Group A — E above 12.6×10^3 N/mm² and f_b above 18.0 N/mm².

Group B — E above 9.8×10^3 N/mm² and up to 12.6×10^3 N/mm² and f_b above 12.0 N/mm² and up to 18.0 N/mm².

Group C — E above 5.6×10^3 N/mm² and up to 9.8×10^3 N/mm² and f_b above 8.5 N/mm² and up to 12.0 N/mm².

NOTE — Modulus of elasticity given above is applicable for all locations and extreme fibre stress in bending is for inside location.

11.4.2

The general characteristics like durability and treat ability of the species are also given in Table 11.4.1.

Species of timber other than those recommended in Table 11.4.1 may be used, provided the basic strength properties are determined and found in accordance with 11.5.1.

NOTE — For obtaining basic stress figures of the unlisted species, reference may be made to the Forest Research Institute, Debra Dun.

11.4.3

The permissible lateral strength (in double shear) of mild steel wire shall be as given in Table 11.4.2 and Table 11.4.3 for different species of timber.

11.4.4

Moisture Content in Timber The permissible moisture content of timber for various positions in buildings shall be as given in Table 11.4.4.

Table 11.4.1 Safe Permissible Stresses for the Species of Timber

Species		Average	Modulus of	Permissible Stress in N/mm ² for Grade I											Preservative Characters		
Botanical Name	Trade Name	Density at 12 percent Content Kg/m ³	Elasticity x 10 ³ N/mm ²	Bending and Tension Along Grains, Extreme Fibre Stress			Shear all Location		Compression Parallel to Grain		Compression Perpendicular to Grain			Durability Class	Treatability Grade	Refracterines to All Seasoning	
				Inside Location	outside Location	wet Location	Horizontal	Along Grain	Inside Location	outside Location	wet Location	Inside Location	outside Location				wet Location
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
<i>Acacia nilotica</i>	Babla	797	—	—	12.9	10.3	1.4	2.1	8.9	7.9	6.4	5.2	4.0	3.3	I	b	B
<i>Aglaia odulis</i>	Aglaia	815	12.56	18.2	15.2	12.1	1.4	2.0	10.1	8.9	7.3	4.4	3.4	2.8	—	—	A
<i>Ailantahus grandis</i>	Gokul	404	7.94	8.3	6.9	5.5	0.6	0.8	5.3	4.7	3.9	1.1	0.9	0.7	III	—	C
<i>Altingia excelsa</i>	Jutili	795	11.37	17.1	14.3	11.4	1.2	1.8	11.0	9.8	8.0	6.8	5.3	4.4	II	e	A
<i>Amoora rehituka</i>	Pitraj	668	8.98	12.3	10.2	8.2	1.1	1.5	8.0	7.1	5.8	4.0	3.1	2.6	I	—	B
<i>Amoora wallichii</i>	Lali	583	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
<i>Amoora spp.</i>	Amari	625	1.05	13.4	1.1	9.2	0.9	1.3	8.4	7.4	6.0	3.7	2.9	2.4	II	d	B
<i>Anisoplera glabra</i>	Boilam	573	—	—	—	—	—	—	—	—	—	—	—	—	III	b	—
<i>Aphenamixis polystachya</i>	Pitraj	583	—	—	—	—	—	—	—	—	—	—	—	—	III	e	B
<i>Arlocarpus chaplasha</i>	Chapalish	515	9.11	13.2	11.0	8.8	0.9	1.2	8.5	7.5	6.2	3.6	2.8	2.3	III	d	B
<i>Artocarpus integrifolia</i>	Kanthai	537	—	—	—	—	—	—	—	—	—	—	—	—	III	c	B
<i>Azadirachta indica</i>	Neem	836	8.52	14.6	12.1	9.7	1.3	1.8	10.0	8.9	7.3	5.0	3.9	3.2	—	—	—
<i>Betula Inoides</i>	Birch	625	9.23	9.6	8.0	6.4	0.8	1.1	5.7	5.0	4.1	2.2	1.7	1.4	—	—	B
<i>Bischofia javanica</i>	Bhadi	769	8.84	9.6	8.2	6.5	0.8	1.1	5.9	5.3	4.3	3.6	2.8	2.3	III	—	A
<i>Bruguiera conjugata</i>	Kankra	879	—	—	—	—	—	—	—	—	—	—	—	—	—	—	A

Species		Average	Modulus of	Permissible Stress in N/mm ² for Grade I											Preservative Characters		
Botanical Name	Trade Name	Density at 12 percent Content Kg/m ³	Elasticity x 10 ³ N/mm ²	Bending and Tension Along Grains, Extreme Fibre Stress			Shear all Location		Compression Parallel to Grain		Compression Perpendicular to Grain			Durability Class	Treatability Grade	Refracterines to All Seasoning	
				Inside Location	outside Location	wet Location	Horizontal	Along Grain	Inside Location	outside Location	wet Location	Inside Location	outside Location				wet Location
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
<i>Bucklandia populnea</i>	Plpli	672	9.89	12.8	10.7	8.6	1.1	1.5	7.9	7.0	5.7	3.5	2.7	2.2	III	e	C
<i>Canarium strictum</i>	White dhup	569	10.54	10.1	8.4	6.7	0.7	1.1	6.2	5.5	4.5	2.1	1.6	1.3	III	—	C
<i>Cassia fistula</i>	Sonalu	865	11.80	19.2	16.0	12.8	1.4	2.0	12.3	10.9	8.9	7.2	5.6	4.6	I	—	A
<i>Castanopsis hystrix</i>	Chestanut	624	9.85	10.6	8.8	7.0	0.8	1.2	6.4	5.7	4.6	2.7	2.1	1.7	II	b	B
<i>Carallia lucida</i>	Maniawaga	748	12.60	18.4	15.3	12.3	1.2	1.7	11.4	10.1	8.3	5.9	4.6	3.8	—	—	—
<i>Cassia siamea</i>	Minjiri	695	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
<i>Chukrasia tabularis</i>	Chickrassy	666	8.35	11.8	9.8	7.9	1.1	1.5	7.1	6.3	5.2	3.9	3.1	2.5	II	c	B
<i>Dalbergia sissoo</i>	Sissoo	808	—	—	—	—	—	—	—	—	—	—	—	—	—	—	B
<i>Dillemia indica</i>	Dillenia	617	8.61	12.1	10.0	8.0	0.8	1.2	7.3	6.5	5.3	2.7	2.1	1.7	III	a	B
<i>Dillenia pentagyne</i>	Dillenia	622	7.56	11.8	9.9	7.9	0.9	1.3	7.1	6.3	5.2	3.5	2.7	2.2	III	d	B
<i>Dipterocarpus alatus</i>	Garjan	721	—	—	—	—	—	—	—	—	—	—	—	—	III	a	B
<i>Dipterocarpus rnacrocarpus</i>	Hollong	726	13.34	14.5	12.0	9.6	0.8	1.1	8.8	7.9	6.4	3.5	2.7	2.2	III	a	B
<i>Duabanga sonneratioides</i>	Banderhol	485	8.38	9.8	8.2	6.5	0.6	0.9	6.4	5.7	4.7	1.8	1.4	1.1	III	c	C
<i>Garuga piannata</i>	Garuga	571	7.58	11.7	9.7	7.8	1.0	1.5	7.2	6.4	5.3	3.4	2.6	2.1	I	e	B
<i>Gerriops roxbarghiana</i>	Goran	869	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
<i>gGmeline arborea</i>	Garnar	501	7.02	9.8	8.2	6.6	0.8	1.4	5.7	5.0	4.1	4.2	3.2	2.7	I	e	B
<i>Grewia veslita</i>	Dhaman	758	12.00	15.4	12.6	10.3	1.4	2.0	9.1	8.1	6.6	4.1	3.2	2.6	III	d	B
<i>Heritiera spp.</i>	Sundri	872	13.37	17.9	14.9	11.9	1.3	1.8	11.0	9.8	8.0	6.5	5.0	4.1	I	—	A

Species		Average Density at 12 percent Content Kg/m ³	Modulus of Elasticity x 10 ³ N/mm ²	Permissible Stress in N/mm ² for Grade I											Preservative Characters		
Botanical Name	Trade Name			Bending and Tension Along Grains, Extreme Fibre Stress			Shear all Location		Compression Parallel to Grain		Compression Perpendicular to Grain			Durability Class	Treatability Grade	Refracterines to All Seasoning	
				Inside Location	outside Location	wet Location	Horizontal	Along Grain	Inside Location	outside Location	wet Location	Inside Location	outside Location				wet Location
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
<i>Hopea odorata</i>	Telsur	711	—	—	—	—	—	—	—	—	—	—	—	—	III	a	B
<i>Kayea floribund</i>	Karal	813	10.88	16.8	14.0	1.1	1.1	1.6	10.1	9.0	7.3	4.4	3.4	2.8	III	—	—
<i>Lagerstrocmia spp.</i>	Jarul	654	—	—	—	—	—	—	—	—	—	—	—	—	III	e	B
<i>Machilus macrantha</i>	Machilus	692	10.00	12.4	10.3	8.3	1.0	1.5	8.2	7.3	6.0	3.5	2.7	2.2	III	e	B/C
<i>Manglietia insignia</i>		449	10.37	10.9	9.1	7.3	0.7	1.4	8.0	7.1	5.8	3.4	2.6	2.1	—	—	—
<i>Manilota polyandra</i>	Ping	903	13.20	19.1	15.9	12.7	1.3	1.8	1.2	10.0	8.5	5.7	4.4	3.6	III	b	A
<i>Mesua assamica</i>	Keyea	842	12.83	17.4	14.5	11.6	1.0	1.4	11.7	10.4	8.5	5.3	4.1	3.3	II	e	—
<i>Mesua ferrea</i>	Mesua	965	16.30	23.3	19.4	15.5	1.2	1.8	15.5	13.8	11.3	5.9	4.6	3.7	I	—	A
<i>Michelia champaca</i>	Champa	644	—	—	—	—	—	—	—	—	—	—	—	—	—	—	B
<i>Michelia montana</i>	Champ	512	8.25	10.9	9.1	7.3	0.7	1.0	6.6	5.9	4.8	2.8	2.2	1.8	I	—	B
<i>Michelia excelsa</i>	Champ	513	10.12	9.8	8.2	6.5	0.7	1.0	6.1	5.5	4.5	1.6	1.3	1.0	II	e	B
<i>Mitragyna pervifolia</i>	Dakroom	651	7.82	12.6	10.5	8.4	1.0	1.5	7.9	7.0	5.7	3.7	2.9	2.4	III	b	B
<i>Palaquium polyanthum</i>	Tali	734	11.24	14.9	12.4	10.0	1.1	1.6	9.9	8.8	7.2	4.7	3.7	3.0	—	—	B
<i>Phoebe hainesiana</i>	Bonsum	566	9.5	13.2	11.0	8.8	0.8	1.2	8.8	7.8	6.4	2.8	2.1	1.8	II	c	B
<i>Phoebe goalperansis</i>	Bonsum	511	7.65	9.7	8.1	6.5	0.7	1.0	6.6	5.9	4.8	2.2	1.7	1.4	II	c	B
<i>Plerygota alata</i>	Narikel	593	10.95	13.4	11.8	8.9	0.8	1.2	8.2	7.3	6.0	2.7	2.1	1.7	III	—	C
<i>Prunus napeulensis</i>	Arupati	548	9.41	4.4	8.7	69.6	0.9	1.2	6.7	6.0	4.9	2.4	1.9	1.6	—	—	—
<i>Pterespermum acerifolium</i>	Hattipaila	607	9.55	13.5	11.3	9.0	0.9	1.2	8.7	7.7	6.3	3.2	2.5	2.0	III	C	B

Species		Average	Modulus of	Permissible Stress in N/mm ² for Grade I											Preservative Characters		
Botanical Name	Trade Name	Density at 12 percent Content Kg/m ³	Elasticity x 10 ³ N/mm ²	Bending and Tension Along Grains, Extreme Fibre Stress			Shear all Location		Compression Parallel to Grain		Compression Perpendicular to Grain			Durability Class	Treatability Grade	Refracterines to All Seasoning	
				Inside Location	outside Location	wet Location	Horizontal	Along Grain	Inside Location	outside Location	wet Location	Inside Location	outside Location				wet Location
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
<i>Quercus lineate</i>	Oak	874	12.63	15.2	12.7	10.1	1.2	1.7	9.6	8.6	7.0	5.3	4.1	3.4	II	c	A
<i>Quercus lamellosa</i>	Oak	87	12.44	14.5	12.1	9.7	1.2	1.7	8.7	7.8	6.4	3.8	2.9	2.4	II	c	A
<i>Schima wallichii</i>	Chilauni	693	9.57	11.1	9.3	7.4	0.9	1.3	6.6	5.9	4.8	2.3	1.8	1.4	III	d	B
<i>Seritiera fomes</i>	Sundri	1073	—	—	—	—	—	—	—	—	—	—	—	—	III	b	B
<i>Shotea assamica</i>	Makai	548	9.27	11.1	9.2	7.4	0.9	1.3	7.1	6.3	5.2	2.9	2.2	1.8	III	c	B
<i>Shorea robusta</i>	Sal	889	—	—	—	—	—	—	—	—	—	—	—	—	III	e	B
<i>Sonneralia apetale</i>	Keora	617	8.63	12.8	10.7	8.5	0.9	1.3	7.4	6.6	5.4	4.8	3.7	3.0	II	—	B
<i>Swintonia floribunda</i>	Civit	665	—	—	—	—	—	—	—	—	—	—	—	—	III	a	C
<i>Syzygium cumini</i>	Jamun	841	10.55	14.8	12.4	9.9	1.1	1.6	9.0	8.0	6.5	6.9	5.4	4.4	II	e	A
<i>Syzygium spp.</i>	Jam	823	—	—	—	—	—	—	—	—	—	—	—	—	III	e	A
<i>Taxus buccata</i>	Yew	705	7.79	14.3	11.9	9.5	1.2	1.7	8.7	7.8	6.4	4.7	3.7	3.0	—	—	—
<i>Tectona grandis</i>	Teak	660	9.97	15.5	12.9	10.3	1.2	1.6	9.4	8.3	6.8	4.5	3.5	2.8	I	e	B
<i>Toena ciliata</i>	Toon	487	6.40	8.7	7.3	5.8	0.7	1.0	5.4	4.8	3.9	2.4	1.8	1.5	II	c	B
<i>Terminalia citrna</i>		755	11.89	17.1	14.3	11.4	1.1	1.6	10.8	9.6	7.9	5.0	3.9	3.2	—	—	—
<i>Terminalia myriocarpa</i>	Hollock	615	9.62	11.9	9.9	8.0	0.9	1.2	7.6	6.7	5.5	2.9	2.2	1.8	III	a	B
<i>Xylia dolabriformis</i>	Lohakat	1007	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
<i>Xylocarpus rolloensis</i>	Passur	757	—	—	—	—	—	—	—	—	—	—	—	—	—	—	B
<i>Zanthoxylum budranga</i>	Mullilam	587	10.65	14.7	12.2	9.8	0.9	1.2	9.5	8.4	6.9	3.4	2.6	2.1	I	e	B

† Classification for preservation based on durability tests, etc.

Class

- I – Average life more than 120 months;
- II – Average life 60 months or above but less than 120 months; and
- III – Average life less than 60 months.

‡ Treatability Grades

- a – Heartwood easily treatable;
- b – Heartwood treatable, but complete penetration not always obtained; in case where the least dimension is more than 60 mm;
- c – Heartwood only partially treatable;
- d – Heartwood refractory to treatment; and
- e – Heartwood very refractory to treatment, penetration of preservative being practically nil even from the ends.

Data based on strength properties at three years of age of tree.

§ Classifications based on seasoning behavior of timber and refractoriness w.r.t. cracking , splitting and drying rate.

- A – Highly refractory (slow and difficulty to season free from surface and end cracking);
 - B – Moderately refractory (may be seasoned free from surface and end cracking within reasonably short periods, given a little protection against rapid drying conditions); and
 - C – Non-refractory (may be rapidly seasoned free from surface and end-cracking even in the open air and sun. If not rapidly dried, they develop blue stain and mould on the surface.
-

Table 11.4.2 Permissible Lateral Strengths (in Double Shear) of Nails 3.55 mm Dia, 80 mm Long

SI No.	Species of Wood		For Permanent Construction Strength per Nail		For Temporary Structures Strength per Nail (for Both Lengthening Joints and Node Joints) Nx10 ²
	Botanical Name	Trade name	Lengthening Joints Nx10 ²	Node Joints Nx10 ²	
1	2	3	4	5	6
	<i>Acacia nilotica</i>	Babla	15	11	34
	<i>Aphenamixis polystachya</i>	Pitraj	19	9	19
	<i>Canarium strictum</i>	White dhup	9	8	10.5
	<i>Castanopsis hystrix</i>	Chestnut	18	10.5	23.5
	<i>Chukrasia tabularis</i>	Chickrassy	24	8	27
	<i>Dillenia pentagyne</i>	Dillenia	16.5	12	16
	<i>Dipterocarpus rnocrocarpus</i>	Hollong	17	7	20
	<i>Grewia veslita</i>	Dhaman	13	5	24
	<i>Hopea odorata</i>	Telsur	31.5	13	28.5
	<i>Lagerstrocmia spp.</i>	Jarul	24.5	21.5	22.5
	<i>Maniltoa polyandra</i>	Ping	26	23.5	32
	<i>Mesua ferrea</i>	Mesua	26	8	41
	<i>Michelia excelsa</i>	Champ	13	9	20
	<i>Phoebe hainesiana</i>	Bonsum	12	6	13
	<i>Shorea robusta</i>	Sal	23	15.5	19.5
	<i>Syzygium spp.</i>	Jam	15	12	25
	<i>Tectona grandis</i>	Teak	14	8	13
	<i>Terminalia myriocarpa</i>	Hollock	13	10	19
	<i>Toona ciliata</i>	Toon	16	9	21

NOTES

1. Nails of 3.55 mm diameter are most commonly used. The above values can also be used for 4 mm diameter 100 mm long nails.
2. The values in N are approximate converted values from kgf. For exact conversion the value is 1 kgf = 9.80665 N.

11.4.4.1 Tolerances

Permissible tolerances in measurements of cut sizes of structural timber shall be as follows:

a) For width and thickness:

1) Up to and including 100 mm $\begin{matrix} +3 \\ -0 \end{matrix}$ mm

2) Above 100 mm $\begin{matrix} +6 \\ -3 \end{matrix}$ mm

b) For length $\begin{matrix} +10 \\ -0 \end{matrix}$ mm

11.4.5 Grading of Structural Timber

11.4.5.1 Cut sizes of structural timber shall be graded, after seasoning, into three grades based on permissible defects given in Table 11.4.8:

- a) Select Grade
- b) Grade I
- c) Grade II

Table 11.4.3 Permissible Lateral Strengths (in Double Shear) of Nails 5.00 mm Dia, 125 mm and 150 mm Long

Species of Wood		For Permanent Construction Strength per Nail		For Temporary Structures Strength per Nail (for Both Lengthening Joints and Node Joints) $\text{Nx}10^2$	
Botanical Name	Trade name	Lengthening Joints $\text{Nx}10^2$	Node Joints $\text{Nx}10^2$		
2	3	4	5	6	
<i>Acacia nilotica</i>	Babla	27	13.5		53
<i>Dalbergia sissoo</i>	Sissoo	17	15		43
<i>Mesua ferrea</i>	Mesua	24	15.5		57.5
<i>Michelia excelsa</i>	Champ	26	12.5		39
<i>Phoebe hainesiana</i>	Bonsum	20	7.5		30
<i>Shorea robusta</i>	Sal	19.5	17		37
<i>Syzygium spp.</i>	Jam	18	14.5		38.5
<i>Tectona grandis</i>	Teak	28	13		30
<i>Terminalia myriocarpa</i>	Hollock	27.5	9		41

Table 11.4.4 Permissible Percentage Moisture Content Values

SI No.	Use	Zones (see Note)			
(1)	(2)	(3)	(4)	(5)	(6)
i)	Structural elements	12	14	17	20
ii)	Doors and windows				
	50 mm and above in thickness	10	12	14	16
	Thinner than 50 mm	8	10	12	14
iii)	Flooring strips for general purposes	8	10	10	12

NOTE — The country has been broadly divided into the following four zones based on the humidity variations in the country:

Zone I — Average annual relative humidity less than 40 percent.

Zone II — Average annual relative humidity 40 to 50 percent.

Zone III — Average annual relative humidity 50 to 67 percent.

Zone IV — Average annual relative humidity more than 67 percent.

For detailed zonal classification, tolerances, etc reference may be made to good practice [6-3A(4)].

11.4.6 Sawn Timber

11.4.6.1 Sizes

Preferred cut sizes of timber for use in structural components shall be as given in Tables 11.4.5 to 11.4.7.

Table 11.4.5 Preferred Cut Sizes of Structural Timbers for Roof Trusses (Span from 3 m to 20 m)

Thickness				Width				
mm				mm				
1	2	3	4	5	6	7	8	9
20	40	50	60	80	100	—	—	—
25	40	50	60	80	100	120	160	180
30	40	50	60	80	100	120	160	180
35	—	—	60	80	100	120	160	180
40	—	—	60	80	100	120	160	180
50	—	—	60	80	100	120	160	180
60	—	—	—	80	100	120	160	180
80	—	—	—	—	100	120	160	180

NOTES

1 For truss spans marginally above 20 m, preferred cut sizes of structural timber may be allowed.

2 Preferred lengths of timber: 1, 1.5, 2, 2.5 and 3 m.

Table 11.4.6 Preferred Cut Sizes of Structural Timber for Roof Purlins, Rafters, Floor Beams, Etc

Thickness				Width			
mm				mm			
1	2	3	4	5	6	7	8
50	80	100	120	140	—	—	—
60	80	100	120	140	160	—	—
80	—	100	120	140	160	—	—
100	—	—	—	140	160	180	200

NOTE —Preferred lengths of timber: 1.5, 2, 2.5 and 3 m.

Table 11.4.7 Preferred Cut Sizes of Structural Timbers for Partition Framing and Covering, and for Centering

Thickness					Width				
mm					mm				
1	2	3	4	5	6	7	8	9	10
10	40	50	60	80	—	—	—	—	—
15	40	50	60	80	100	—	—	—	—
20	40	50	60	80	100	120	160	200	—
25	40	50	60	80	100	120	160	200	240
30	40	50	60	80	100	120	160	200	240
40	40	—	60	80	100	120	160	200	240
50	—	50	—	80	100	120	160	200	240
60	—	—	60	80	100	120	160	200	240
80	—	—	—	80	100	120	160	200	240

Table 11.4.8 Permissible Defects for Cut Sizes of Timber for Structural Use

SI No.	Defects	Select Grade	Grade I		Grade II		
1	2	3	4		5		
i)	Wane	Shall be permissible at its deepest portion up to a limit of 1/8 of the width of the surface on which it occurs	Shall be permissible at its deepest portion up to a limit of 1/6 of the width of the surface on which it occurs		Shall be permissible at its deepest portion up to a limit of 1/4 of the width of the surface on which it occurs		
ii)	Worm holes	Other than those due to powder post beetles are permissible	Other than those due to powder post beetles are permissible		Other than those due to powder post beetles are permissible		
iii)	Slope of grain	Shall not be more than 1 in 20	Shall not be more than 1 in 15		Shall not be more than 1 in 12		
iv)	Live knots:						
	Width of Wide Faces of Cut Sizes of Timber	Permissible Maximum Size of Live Knot on	Permissible Maximum Size of Live Knot on		Permissible Maximum Size of Live Knot on		
	Max	Narrow faces and 1/4 of the width face close to edges of cut size of timber	Remaining central half of the width of the wide faces	Narrow faces and 1/4 of the width face close to edges of cut size of timber	Narrow faces and 1A of the width face close to edges of cut size of timber	Remaining central half of the width of the wide faces	
	1	2	3	4	5	6	7
	75	10	10	19	19	29	30
	100	13	13	25	25	38	39
	150	19	19	38	38	57	57
	200	22	25	44	50	66	75
	250	25	29	50	57	66	87
	300	27	38	54	75	81	114
	350	29	41	57	81	87	123
	400	32	44	63	87	96	132
	450	33	47	66	93	99	141
	500	35	50	69	100	105	150
	550	36	52	72	103	108	156
	600	38	53	75	106	114	159
v)	Checks and shakes:						

Width of the Face of the Timber Max	Permissible Depth Max	Permissible Depth Max	Permissible Depth Max
1	2	3	4
75	12	25	36
100	18	35	54
150	25	50	75
200	33	65	99
250	40	81	120
300	50	100	150
350	57	115	171
400	66	131	198
450	76	150	225
500	83	165	249
550	90	181	270
600	100	200	300

11.4.6.2 The prohibited defects given in 4.6.2.1 and permissible defects given in 4.6.2.2 shall apply to structural timber.

11.4.6.2.1 Prohibited defects

Loose grains, splits, compression wood in coniferous species, heartwood rot, sap rot, crookedness, worm holes made by powder post beetles and pitch pockets shall not be permitted in all the three grades.

11.4.6.2.2 Defects to the extent specified in Table 11.4.8 shall be permissible.

NOTE — Wanes are permitted provided they are not combined with knots and the reduction in strength on account of the wanes is not more than the reduction with maximum allowable knots.

11.4.6.3 Location of Defects

The influence of defects in timber is different for different locations in the structural element. Therefore, these should be placed during construction in such a way so that they do not have any adverse effect on the members.

11.4.7 Suitability

11.4.7.1 Suitability in Respect of Durability and Treatability for Permanent Structures

There are two choices as given in 11.4.7.1.1 and 11.4.7.1.2.

11.4.7.1.1 First choice

The species shall be any one of the following:

- Untreated heartwood of high durability. Heartwood if containing more than 15 percent sap wood, may need chemical treatment for protection;
- Treated heartwood of moderate and low durability and class 'a' and class 'b' treatability;
- Heartwood of moderate durability and class 'c' treatability after pressure impregnation, and
- Sapwood of all classes of durability after thorough treatment with preservative.

11.4.7.1.2 Second choice

The species of timber shall be heartwood of moderate durability and class 'd' treatability.

11.4.7.2 Choice of load-bearing temporary structures or semi-structural components at construction site

- Heartwood of low durability and class 'e' treatability; or

- b) The species whose durability and/or treatability are yet to be established, as listed in Table 11.4.1.

11.4.8 Fastenings

All structural members shall be framed, anchored, tied and braced to develop the strength and rigidity necessary for the purposes for which they are used.

Allowable stresses or loads on joints and fasteners shall be determined in accordance with recognized principles. Common mechanical fastenings are of bar type such as nails and spikes, wood screws and bolts, and timber connectors including metallic rings or wooden disc-dowels. Chemical fastenings include synthetic adhesives for structural applications.

11.5 PERMISSIBLE STRESSES

11.5.1 The permissible stresses for Groups A, B and C for different locations applicable to Grade I structural timber shall be as given in Table 11.5.1 provided that the following conditions are satisfied:

- The timbers should be of high or moderate durability and be given the suitable treatment where necessary.
- Timber of low durability shall be used after proper preservative treatment and
- The loads should be continuous and permanent and not of impact type.

Table 11.5.1 Minimum Permissible Stress Limits (N/mm²) in Three Groups of Structural Timbers (for Grade I Material)

SI No.	Strength Character	Location of Use	Group A	Group B	Group C
(1)	(2)	(3)	(4)	(5)	(6)
i)	Bending and tension along grain	Inside ¹⁾	18.0	12.0	8.5
ii)	Shear ²⁾	All locations	1.05	0.64	0.49
	Horizontal				
	Along grain	All locations	15	0.91	0.70
iii)	Compression perpendicular to grain	Inside ¹⁾	11.7	7.8	4.9
iv)	Compression perpendicular to grain	Inside ¹⁾	4.0	2.5	1.1
v)	Modulus of elasticity (×10 ³ N/mm ²)	All locations and grade	12.6	9.8	5.6

- For working stresses for other locations of use, that is, outside and wet, generally factors of 5/6 and 2/3 are applied.
- The values of horizontal shear to be used only for beams. In all other cases shear along grain to be used.

11.5.2 The permissible stresses (excepting E) given in Table 11.5.1 shall be multiplied by the following factors to obtain the permissible stresses for other grades provided that the conditions laid down in 5.2 are satisfied:

- For Select Grade Timber 1.16
- For Grade II Timber 0.84

When low durability timbers are to be used [see 5.2(b)] on outside locations, the permissible stresses for all grades of timber, arrived at by 5.2 and 5.3 shall be multiplied by 0.80.

11.5.3 Modification Factors for Permissible Stresses

11.5.3.1 Due to Change in Slope of Grain

When the timber has not been graded and has major defects like slope of grain, knots and checks or shakes but not beyond permissible value, the permissible stress given in Table 11.4.1 shall be multiplied by modification factor K_1 for different slopes of grain as given in Table 11.5.2.

Table 11.5.2 Modifications Factor K_1 to Allow for Change in Slope of Grain

slope	Modification Factor K_1	
	Strength of Beams, Joists and Ties	Strength of Posts or Columns
1	2	3
1 in 10	0.80	0.74
1 in 12	0.90	0.82
1 in 14	0.98	0.8
1 in 15 and flatter	1.00	1.00

NOTE — For intermediary slopes of grains, values of modification factor may be obtained by interpolation.

11.5.3.2 Due to Duration of Load

For different durations of design load, the permissible stresses given in Table 1 shall be multiplied by the modification factor K_2 given in Table 11.5.3.

Table 11.5.3 Modifications Factor K_2 , for Change in Duration of Loading

Duration of Loading	Modification Factor K_2
1	2
Continuous (Normal)	1.0
Two months	1.15
Seven days	1.25
Wind and earthquake	1.33
Instantaneous or impact	2.00

NOTE — The strength properties of timber under load are time dependent.

- 11.5.3.2.1 The factor K_2 is applicable to modulus of elasticity when used to design timber columns, otherwise they do not apply thereto.
- 11.5.3.2.2 If there are several duration of loads (in addition to the continuous) to be considered, the modification factor shall be based on the shortest duration load in the combination, that is, the one yielding the largest increase in the permissible stresses, provided the designed section is found adequate for a combination of other larger duration loads.
[Explanation: In any structural timber design for dead loads, snow loads and wind or earthquake forces, members may be designed on the basis of total of stresses due to dead, snow and wind loads using $K_2 = 1.33$, factor for the permissible stress (of Table 11.4.1) to accommodate the wind load, that is, the shortest of duration and giving the largest increase in the permissible stresses. The section thus found is checked to meet the requirements based on dead loads alone with modification $K_2 = 1.00$].
- 11.5.3.2.3 Modification factor K_2 shall also be applied to allowable loads for mechanical fasteners in design of joints, when the wood and not the strength of metal determine the load capacity.

11.6 DESIGN CONSIDERATIONS

11.6.1 All structural members, assemblies or framework in a building, in combination with the floors, walls and other structural parts of the building shall be capable of sustaining, with due stability and stiffness the whole dead and imposed loadings as per Part 6 'Structural Design, Section 1 Loads, Forces and Effects', without exceeding the limits of relevant stresses specified in this Section.

11.6.2 Buildings shall be designed for all dead and imposed loads or forces assumed to come upon them during construction or use, including uplifts or horizontal forces from wind and forces from earthquakes or other loadings. Structural members and their connections shall be proportioned to provide a sound and stable structure with adequate strength and stiffness. Wooden components in construction generally include panels for sheathing and diaphragms, siding, beams, girder, columns, light framings, masonry wall and joist construction, heavy-frames, glued laminated structural members, structural sandwiches, prefabricated panels, lamella arches, portal frames and other auxiliary constructions.

11.6.3 Net Section

11.6.3.1 The net section is obtained by deducting from the gross sectional area of timber the projected area of all material removed by boring, grooving or other means at critical plane. In case of nailing, the area of the prebored hole shall not be taken into account for this purpose.

11.6.3.2 The net section used in calculating load carrying capacity of a member shall be at least net section determined as above by passing a plane or a series of connected planes transversely through the members.

11.6.3.3 Notches shall be in no case remove more than one quarter of the section.

11.6.3.4 In the design of an intermediate or a long column, gross section shall be used in calculating load carrying capacity of the column.

11.6.4 Loads

11.6.4.1 The loads shall conform to those given in Part 6 Structural Design, Chapter 2 Loads on Buildings and Structures.

11.6.4.2 The worst combination and location of loads shall be considered for design. Wind and seismic forces shall not be considered to act simultaneously.

11.6.5 Flexural Members

11.6.5.1 Such structural members shall be investigated for the following:

- a) Bending strength,
- b) Maximum horizontal shear,
- c) Stress at the bearings, and
- d) Deflection.

11.6.5.2 Effective Span

The effective span of beams and other flexural members shall be taken as the distance from face of supports plus one-half of the required length of bearing at each end except that for continuous beams and joists the span may be measured from centre of bearing at those supports over which the beam is continuous.

11.6.5.3 Usual formula for flexural strength shall apply in design:

$$f_{ab} \frac{M}{Z} \leq f_b$$

11.6.5.4 Form Factors for Flexural Members

The following form factors shall be applied to the bending stress:

- a) Rectangular Section — For rectangular sections, for different depths of beams, the form factor K_3 shall be taken as:

$$K_3 = 0.81 \left(\frac{D^2 + 89400}{D^2 + 55000} \right)$$

NOTE—Form factor (K_3) shall not be applied for beams having depth less than or equal to 300 mm.

- b) Box Beams and I-Beams — For box beams and I-beams, the form factor K_A obtained by using the formula:

$$K_4 = 0.8 + 0.8y \left(\frac{D^2 + 89400 - 1}{D^2 + 55000} \right)$$

where

$$y = p_1^2 + (6 - 8p_1 + 3p_1^2)(1 - q_1) + q_1$$

- c) Solid Circular Cross-Sections — For solid circular cross sections the form factor K_S shall be taken as 1.18.
- d) Square Cross-Sections — For square cross-sections where the load is in the direction of diagonal, the form factor K_G shall be taken as 1.414.

11.6.5.5 Width

The minimum width of the beam or any flexural member shall not be less than 50 mm or 1/50 of the span, whichever is greater.

11.6.5.6 Depth

The depth of beam or any flexural member shall not be taken more than three times of its width without lateral stiffening.

11.6.5.6.1 Stiffening

All flexural members having a depth exceeding three times its width or a span exceeding 50 times its width or both shall be laterally restrained from twisting or buckling and the distance between such restraints shall not exceed 50 times its width.

11.6.5.7 Shear

11.6.5.7.1 The following formulae shall apply:

- a) The maximum horizontal shear, when the load on a beam moves from the support towards the centre of the span, and the load is at a distance of three to four times the depth of the beam from the support, shall be calculated from the following general formula:

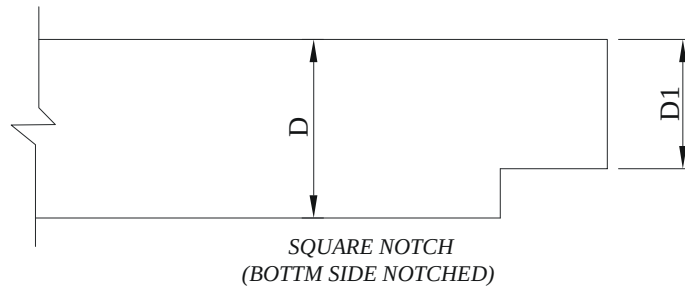
$$H = \frac{VQ}{Ib}$$

- b) For rectangular beams:

$$H = \frac{3V}{2bD}$$

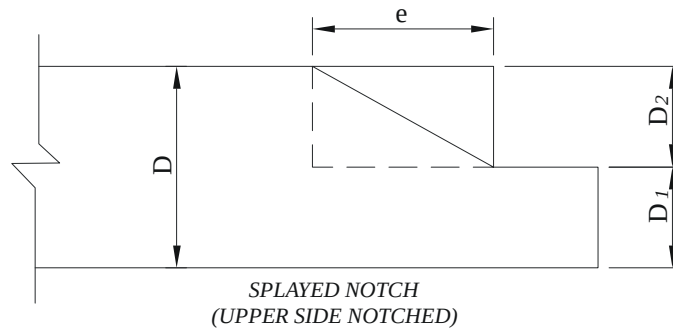
- c) For notched beams, with tension notch at supports:

$$H = \frac{3VD}{2bD_1^2}$$



- d) For notched at upper (compression) face, where $e > D$:

$$H = \frac{3V}{2bD_1}$$



- e) For notched at upper (compression) face, where $e < D$

$$H = \frac{3V}{2b \left[D - \left(\frac{D_2}{D} \right) e \right]}$$

11.6.5.7.2 For concentrated loads:

$$V = \frac{10C(I - x)(x/D)^2}{9I[2 + (x/D)^2]}$$

and for uniformly distributed loads,

$$V = \frac{W}{2} \left(1 - \frac{2D}{I} \right)$$

After arriving at the value of V, its value will be substituted in the formula:

$$H = \frac{VQ}{Ib}$$

11.6.5.7.3 In determining the vertical reaction following deductions in loads maybe made:

- Consideration shall be given to the possible distribution of load to adjacent parallel beams, if any;
- All uniformly distributed loads within a distance equal to the depth of the beam from the edge of the nearest support may be neglected except in case of beam hanging downwards from a particular support, and
- All concentrated loads in the vicinity of the supports may be reduced by the reduction factor applicable according to Table 11.6.1.

Table 11.6.1 Reduction Factor for Concentrated Loads in the Vicinity of Supports

Distance of Load from the Nearest Support	LSD or Less	2D	2.5D	3D or More
1	2	3	4	5
Reduction factor	0.6	0.4	0.2	No Reduction

NOTE — For intermediate distances, factor may be obtained by linear interpolation.

11.6.5.7.4 Unless the local stress is calculated and found to be within the permissible stress, flexural member shall not be cut, notched or bored except as follows:

- Notches may be cut in the top or bottom neither deeper than one-fifth of the depth of the beam nor farther from the edge of the support than one-sixth of the span;
- Holes not larger in diameter than one quarter of the depth maybe bored in the middle third of the depth and length; and
- If holes or notches occur at a distance greater than three times the depth of the member from the edge of the nearest support, the net remaining depth shall be used in determining the bending strength.

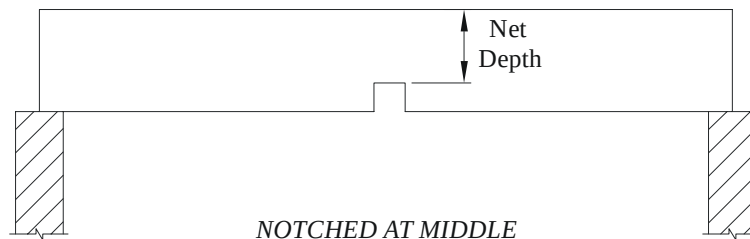


Fig.11.6.1 Notched Beams

11.6.5.8 Bearing

11.6.5.8.1 The ends of flexural members shall be supported in recesses which provide adequate ventilation to prevent dry rot and shall not be enclosed. Flexural members except roof timbers which are supported directly on masonry or concrete shall have a length of bearing of not less than 75 mm. Members supported on corbels, offsets and roof timbers on a wall shall bear immediately on and be fixed to wall plate not less than 75 mm x 40 mm.

11.6.5.8.2 Timber joists or floor planks shall not be supported on the top flange of steel beams unless the bearing stress, calculated on the net bearing as shaped to fit the beam, is less than the permissible compressive stress perpendicular to the grain.

11.6.5.8.3 Bearing stress

Length and position of bearing

- At any bearing on the side grain of timber, the permissible stress in compression perpendicular to the grain, f_{cn} , is dependent on the length and position of the bearing.
- The permissible stresses given in Table 11.4.1 for compression perpendicular to the grain are also the permissible stresses for any length at the ends of a member and for bearings 150 mm or more in length at any other position.
- For bearings less than 150 mm in length located 75 mm or more from the end of a member as shown in Fig. 11.6.2, the permissible stress may be multiplied by the modification factor K_7 given in Table 11.6.2.
- No allowance need be made for the difference in intensity of the bearing stress due to bending of a beam.
- The bearing area should be calculated as the net area after allowance for the amount of wane.
- For bearings stress under a washer or a small plate, the same coefficient specified in Table 11.6.2 may be taken for a bearing with a length equal to the diameter of the washer or the width of the small plate.
- When the direction of stress is at angle to the direction of the grain in any structural member, then the permissible bearing stress in that member shall be calculated by the following formula:

$$f_c \theta = \frac{f_{cp} \times f_{cn}}{f_{cp} \sin^2 \theta + f_{cn} \cos^2 \theta}$$

Table 11.6.2: Modification factor K_7 for bearing stresses

Length of bearing in mm	15	25	40	50	75	100	150 or more
Modification factor K_7	1.67	1.40	1.25	1.20	1.13	1.10	1.00

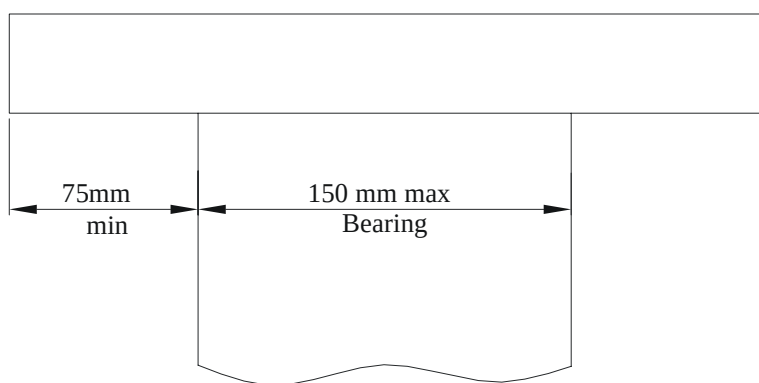


Fig. 11.6.2 Position Of End Bearings

11.6.5.9 Deflection

The deflection in the case of all flexural members supporting brittle materials like gypsum ceilings, slates, tiles and asbestos sheets shall not exceed 1/360 of the span. The deflection in the case of other flexural members shall not exceed 1/240 of the span and 1/150 of the freely hanging length in the case of cantilevers.

11.6.5.9.1

Usual formula for deflection shall apply:

$$\delta = \frac{KWL^3}{EI} \quad (\text{ignoring deflection due to shear strain})$$

K-values = 1/3 for cantilevers with load at free end,
1/8 for cantilevers with uniformly distributed load,
1/48 for beams supported at both ends with point load at centre, and
5/384 for beams supported at both ends with uniformly distributed load.

11.6.5.9.2 In order to allow the effect of long duration loading on E, for checking deflection in case of beams and joists the effective loads shall be twice the dead load if timber is initially dry.

11.6.5.9.3 Self weight of beam shall be considered in design.

11.6.6 Columns

NOTE — The formulae given are for columns with pin end conditions and the length shall be modified suitably with other end conditions.

11.6.6.1 Solid Columns

Solid columns shall be classified into short, intermediate and long columns depending upon their slenderness ratio (S/d) as follows:

- a) Short columns — where S/d does not exceed 11.
- b) Intermediate columns — where S/d is between 11 and K_g , and
- c) Long columns — where S/d is greater than K_g .

11.6.6.1.1 For short columns, the permissible compressive stress shall be calculated as follows:

$$f_c = f_{cp}$$

11.6.6.1.2 For intermediate columns, the permissible compressive stress is calculated by using the following formula:

$$f_c = f_{cp} \left[1 - \frac{1}{3} \left(\frac{S}{K_g d} \right)^4 \right]$$

11.6.6.1.3 For long columns, the permissible compressive stress shall be calculated by using the following formula:

$$f_c = \frac{0.329E}{(S/d)^2}$$

11.6.6.1.4 In case of solid columns of timber, S/d ratio shall not exceed 50.

11.6.6.1.5 The permissible load on a column of circular cross-section shall not exceed that permitted for a square column of an equivalent cross-sectional area.

11.6.6.1.6 For determining S/d ratio of a tapered column, its least dimension shall be taken as the sum of the corresponding least dimensions at the small end of the column and one-third of the difference between this least dimension at the small end and the corresponding least dimension at the large end, but in no case shall the least dimension for the column be taken as more than one and a half times the least dimension at the small end. The induced stress at the small end of the tapered column shall not exceed the permissible compressive stress in the direction of grain.

11.6.6.2 Built-up Columns

11.6.6.2.1 Box column

Box columns shall be classified into short, intermediate and long columns as follows:

- a) Short columns — where $\frac{S}{\sqrt{d_1^2 + d_2^2}}$ is less than 8;
- b) Intermediate columns — where $\frac{S}{\sqrt{d_1^2 + d_2^2}}$ is between 8 and K_9 ; and
- c) Long columns — where $\frac{S}{\sqrt{d_1^2 + d_2^2}}$ is greater than K_9 .

11.6.6.2.2 For short columns, the permissible compressive stress shall be calculated as follows:

$$f_c = qf_{cp}$$

11.6.6.2.3 For intermediate columns, the permissible compressive stress shall be obtained using the following formula

$$f_c = qf_{cp} \left[1 - \frac{1}{3} \left(\frac{S}{\sqrt{d_1^2 + d_2^2}} \right)^4 \right]$$

11.6.6.2.4 For long columns, the permissible compressive stress shall be calculated by using the following formula:

$$f_c = \frac{0.329UE}{\left(\frac{S}{\sqrt{d_1^2 + d_2^2}} \right)^2}$$

11.6.6.2.5 The following values of U and q, depending upon plank thickness (t) in 11.6.6.2.3 and 11.6.6.2.4, shall be used:

t (mm)	U	q
25	0.80	1.00
30	0.60	1.00

11.6.6.3 Spaced Columns

11.6.6.3.1 The formulae for solid columns as specified in 6.6.1 are applicable to spaced columns with a restraint factor of 2.5 or 3, depending upon distances of end connectors in the column.

NOTE — A restrained factor of 2.5 for location of centroid group of fasteners at $S/20$ from end and 3 for location at $S/10$ to $S/20$ from end shall be taken.

11.6.6.3.2 For intermediate spaced column, the permissible compressive stress shall be:

$$f_c = f_{cp} \left[1 - \frac{1}{3} \left(\frac{S}{k_{10}d} \right)^4 \right]$$

11.6.6.3.3 For long spaced columns, the formula shall be:

$$f_c = \frac{0.329E \times 2.5}{(S/d)^2}$$

11.6.6.3.4 For individual members of spaced columns, S/d ratio shall not exceed 80.

11.6.6.4 Compression members shall not be notched. When it is necessary to pass services through such a member, this shall be effected by means of a bored hole provided that the local stress is calculated and found to be within the permissible stress specified. The distance from the edge of the hole to the edge of the member shall not be less than one quarter of width of the face.

11.6.7 Structural Members Subject to Bending and Axial Stresses

11.6.7.1 Structural members subjected both to bending and axial compression shall be designed to comply with the following formula:

$$\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b}$$

11.6.7.2 Structural members subjected both to bending and axial tension shall be designed to comply with the following formula:

$$\frac{f_{at}}{f_t} + \frac{f_{ab}}{f_b}$$

11.7 DESIGN OF COMMON STEEL WIRE NAIL JOINTS

11.7.1 General

Nail jointed timber construction is suitable for light and medium timber framings (trusses, etc) up to 15 m spans. With the facilities of readily available materials and simpler workmanship in mono-chord and split chord constructions, this type of fabrication has a large scope.

11.7.2 Dimensions of Members

11.7.2.1 The dimension of art individual piece of timber (that is, any single member) shall be within the range given below:

- a) The minimum thickness of the main members in mono-chord construction shall be 30 mm.
- b) The minimum thickness of an individual piece of members in split-chord construction shall
- c) The space between two adjacent pieces of timber shall be restricted to a maximum of 3 times the thickness of the individual piece of timber of the chord member. In case of web members, it may be greater for joining facilities.

11.7.3 No lengthening joint shall preferably be located at a panel point. Generally not more than two, but preferably one, lengthening joint shall be permitted between the two panel points of the members.

11.7.4 Specification and Diameter of Nails

11.7.4.1 The nails used for timber joints shall conform to Part 5 'Building Materials'. The nails shall be diamond pointed.

11.7.4.2 The diameter of nail shall be within the limits of one-eleventh to one-sixth of the least thickness of members being connected.

11.7.4.3 Where the nails are exposed to be saline conditions, common wire nails shall be galvanized.

11.7.5 Arrangement of Nails in the Joints

The end distances, edge distances and spacings of nails in a nailed joint should be such as to avoid undue splitting of the wood and shall not be less than those given in 11.7.5.1 and 11.7.5.2.

11.7.5.1 Lengthening Joints

The requirement of spacing of nails in a lengthening joint shall be as follows (see also Fig. 11.7.1):

Sl. No.	Spacing of Nails	Type of Stress in the Joint	Requirement Min
(1)	(2)	(3)	(4)
i)	End distance	Tension	12n
		Compression	10n
ii)	In direction of grain	Tension	10n
		Compression	5n
iii)	Edge distance	-	5n
iv)	Between row of nails perpendicular to the grain	-	5n
Notes:			
1. n is shank diameter of nails			
2. The 5n distance between the rows of nails perpendicular to the grain may be increased subject to the availability of width of the member keeping edge distance constant.			

11.7.5.2 Node Joints

The requirement for spacing of nails in node joints shall be as specified in Fig. 11.7.2 where the members are at right angle and as in Fig. 11.7.3 where the members are inclined to one another at angles other than 90° and subjected to either pure compression or pure tension.

11.7.6 Penetration of Nails

11.7.6.1 For a lap joint when the nails are driven from the side of the thinner member, the length of penetration of nails in the thicker member shall be one and a half times the thickness of the thinner member subject to maximum of the thickness of the thicker member.

11.7.6.2 For butt joints the nails shall be driven through the entire thickness of the joint.

11.7.7 Design Considerations

11.7.7.1 Where a number of nails are used in a joint, the allowable load in lateral resistance shall be the sum of the allowable loads for the individual nails, provided that the centroid of the group of these nails lies on the axis of the member and the spacings conform to 7.5. Where a large number of nails are to be provided at a joint, they should be so arranged that there are more of rows rather than more number of nails in a row.

11.7.7.2 Nails shall, as far as practicable, be arranged so that the line of force in a member passes through the centroid of the group of nails. Where this is not practicable, allowance shall be made for any eccentricity in computing the maximum load on the fixing nails as well as the loads and bending moment in the member.

11.7.7.3 Adjacent nails shall preferably be driven from opposite faces, that is, the nails are driven alternatively from either face of joint.

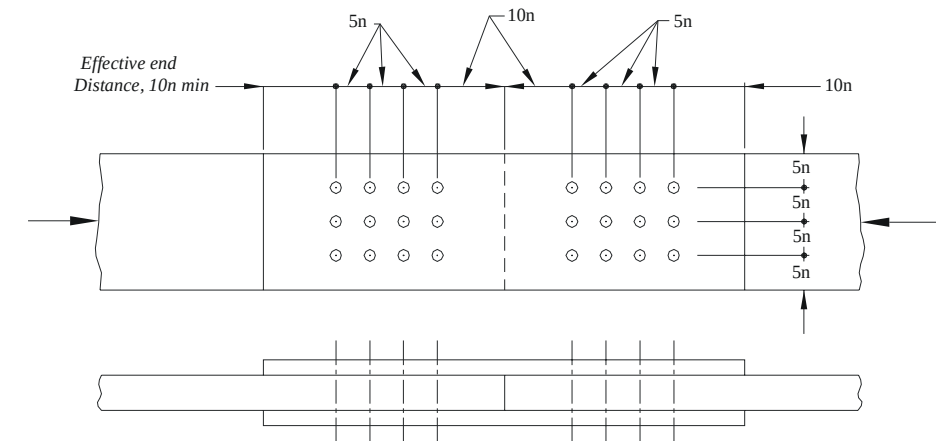
11.7.7.4 For a rigid joint, a minimum of 2 nails for nodal joints and 4 nails for lengthening joint shall be driven.

11.7.7.5 Two nails in a horizontal row are better than using the same number of nails in a vertical row.

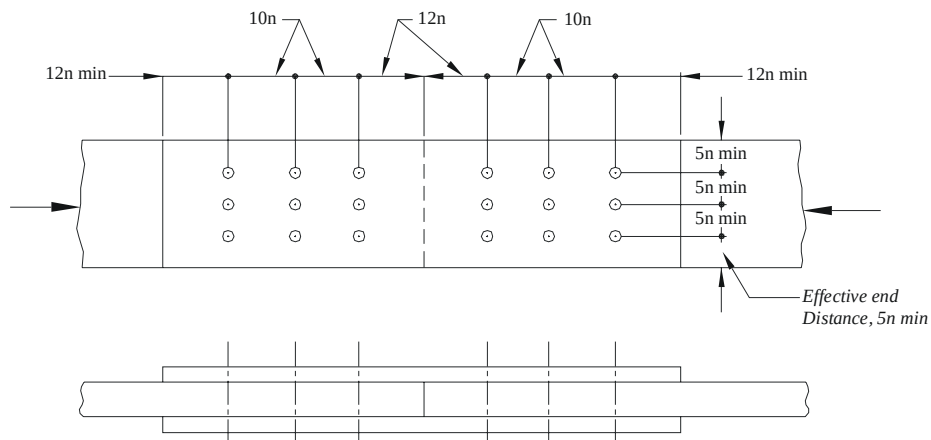
11.7.8 Special Consideration in Nail-Jointed Truss Construction

11.7.8.1 The initial upward camber provided at the centre of the lower chord of nail-jointed timber trusses shall be not less than $1/200$ of the effective span for timber structures using seasoned wood and $1/100$ for unseasoned or partially seasoned wood.

11.7.8.2 The total combined thickness of the gusset or splice plates on either side of the joint in a mono-chord type construction shall not be less than one and a half times the thickness of the main members subject to a minimum thickness of 25 mm of individual gusset plate.



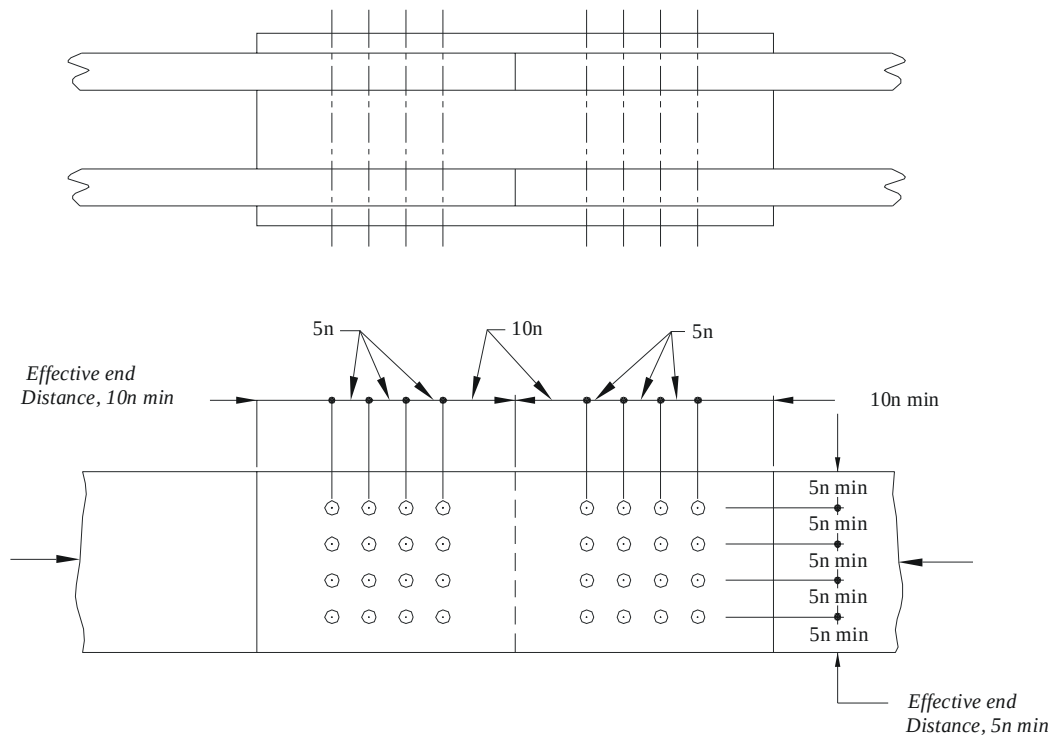
2A MONOCHORD TYPE BUTT JOINT SUBJECT TO COMPRESSION



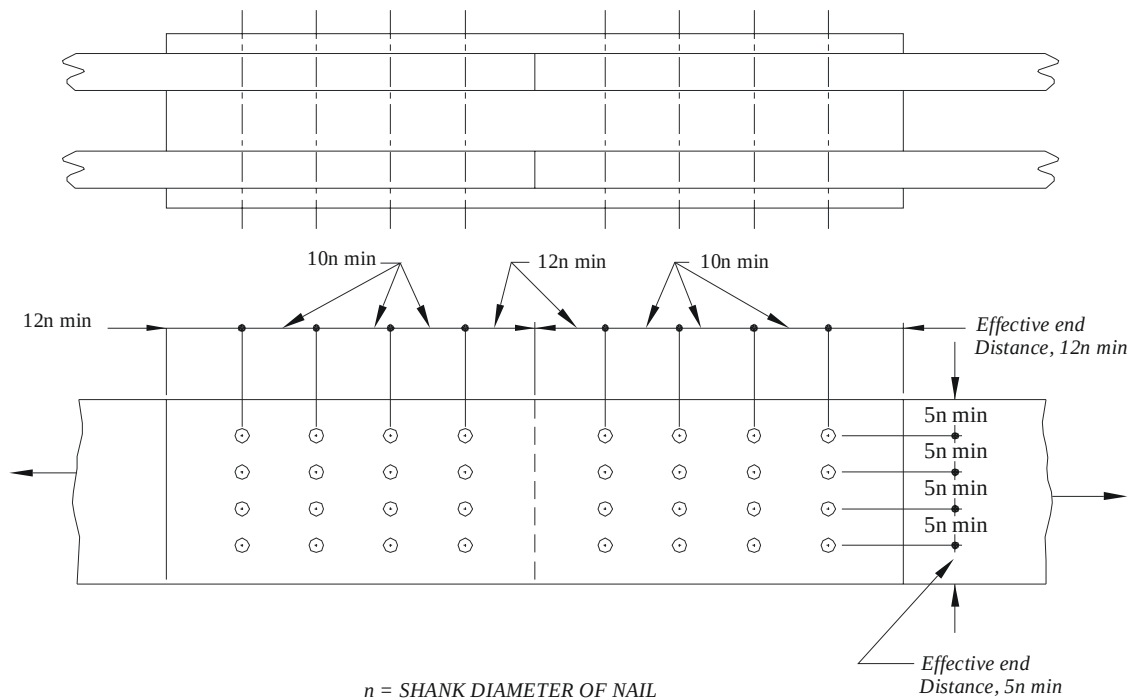
n = SHANK DIAMETER OF NAIL

2B MONOCHORD TYPE BUTT JOINT SUBJECT TO TENSION

Fig. 11.7.1 Spacing of Nails In A Lengthening Joint - Continued



2C SPLIT - CHORD TYPE BUTT JOINT SUBJECT TO COMPRESSION



2C SPLIT - CHORD TYPE BUTT JOINT SUBJECT TO TENSION

Fig. 11.7.1 Spacing of Nails In A Lengthening Joint

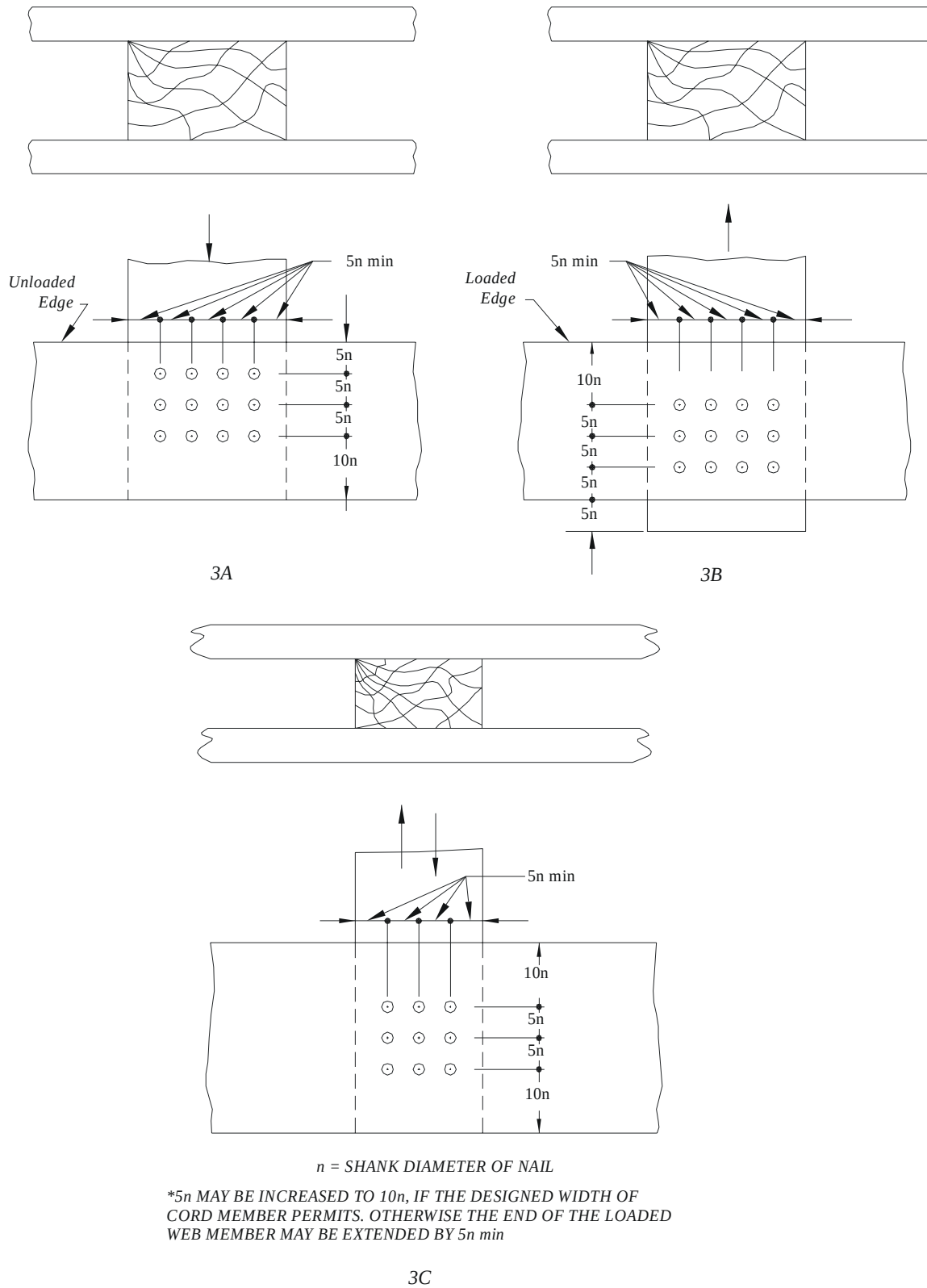
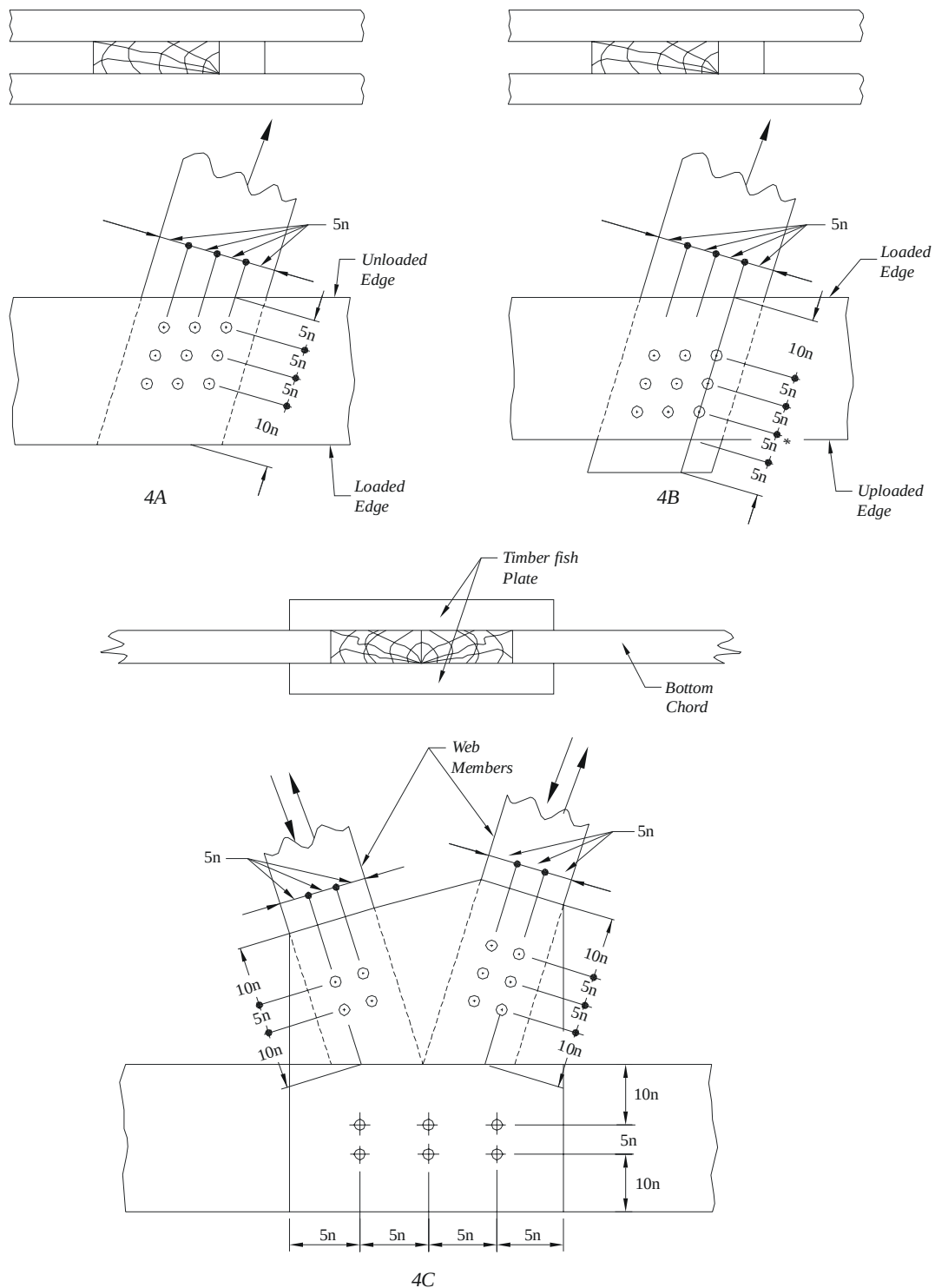


Fig. 11.7.2 Spacing Of Nails Where Members Are At Right Angles To One Another



* $5n$ MAY BE INCREASED TO $10n$, IF THE DESIGNED WIDTH OF CHORD MEMBER PERMITS.
 OTHERWISE THE END OF THE LOADED WEB MEMBER MAY BE EXTENDED BY $5n$ min
 n = SHANK DIAMETER OF NAIL

Fig. 11.7.3 Spacing Of Nails At Node Where Members Are Inclined To One Another

- 11.7.8.3 The total combined thickness of all spacer blocks or plates or both including outer splice plates, at any joint in a split-chord type construction shall not be less than one and a half times the total thickness of all the main members at that joint.

11.8 DESIGN OF NAIL LAMINATED TIMBER BEAMS

11.8.1 Method of Arrangement

- 11.8.1.1 The beam is made up of 20 mm to 30 mm thick planks placed vertically with joints staggered in the adjoining planks with a minimum distance of 300 mm. The planks are laminated with the help of wire nails at regular intervals to take up horizontal shear developed in the beam besides keeping the planks in position (see Fig. 11.8.1).
- 11.8.1.2 The advantage in laminations lies in dimensional stability, dispersal of defects and better structural performance.

11.8.2 Sizes of Planks and Beams

- 11.8.2.1 The plank thickness for fabrication of nailed laminated beams recommended are 20,25 and 30 mm.
- 11.8.2.2 In case of nailed laminated timber beam the maximum depth and length of planks shall be limited to 250 mm and 2000 mm, respectively.
- 11.8.2.3 In order to obtain the overall width of the beam, the number and thickness of planks to form vertical nailed laminated beams, and also type and size of wire nail shall be as mentioned in Table 10.8.1. The protruding portion of the nail shall be cut off or clenched across the grains.

11.8.3 Design Considerations

- 11.8.3.1 Nail laminated beams shall be designed in accordance with 11.6.

Table 10.8.1: Number and size of planks and nails for nailed laminated beams

Sl. No.	Overall Width of Beam (mm)	No. of Planks	Thickness of each Plank (mm)	No. and Size of Nail to be used (mm)
(1)	(2)	(3)	(4)	(5)
i)	50	2	25	80 long 3.55 dia
ii)	60	3	20	-do-
iii)	70	3	(2x25) (1x20)	-do-
iv)	80	4	20	100 long 4.0 dia
v)	90	3	30	-do-
vi)	100	4	25	125 long 5.0 dia
vii)	110	4	(3x30) (1x20)	-do-
viii)	120	4	30	-do-
ix)	150	5	30	150 long 5.0 dia

Notes: A number of combinations of different thickness of planks may be adopted as long as the minimum and maximum thickness of the planks are adhered to.

- 11.8.3.1.1 The deflection in the case of nailed laminated timber beams, joists, purlins, battens and other flexural members supporting brittle materials like gypsum, ceiling slates, tiles and asbestos sheets shall not exceed $1/480$ of the span. The deflection in case of other flexural members shall not exceed $1/360$ of the span in the case of beams and joists, and $1/225$ of the freely hanging length in case of cantilevers.
- 11.8.3.2 Permissible lateral strength of mild steel wire nails shall be as given in Table 11.4.2 and Table 11.4.3 for Indian Species of timber, which shall apply to nails that have their points cut flush with the faces. For nails clenched across the grains the strength may be increased by 20 percent over the values for nails with points cut flush.
- 11.8.3.3 **Arrangement of Nails**
- 11.8.3.3.1 A minimum number of four nails in a vertical row at regular interval not exceeding 75 mm to take up horizontal shear as well as to keep the planks in position shall be used. Near the joints of the planks this distance may, however, be limited to 5 cm instead of 75 mm.
- 11.8.3.3.2 Shear shall be calculated at various points of the beam and [he number of nails required shall be accommodated within the distance equal to the depth of the beam, with a minimum of 4 nails in a row at a standard spacing as shown in Fig. 11.8.2.
- 11.8.3.3.3 If the depth of the beam is more, then the vertical intermediate spacing of nails may be increased proportionately.
- 11.8.3.3.4 If the nails required at a point are more than that can be accommodated in a row, then these shall be provided lengthwise of the beam within the distance equal to the depth of the beam at standard lengthwise spacing.
- 11.8.3.3.5 For nailed laminated beam minimum depth of 100 mm for 3.55 mm and 4 mm diameter nails, and 125 mm for 5 mm diameter nails shall be provided.

11.9 DESIGN OF BOLTED CONSTRUCTION JOINTS

11.9.1 General

Bolted joints suit the requirements of prefabrication in small and medium span timber structures for speed and economy in construction. Bolt jointed construction units offer better facilities as regards to workshop ease, mass production of components, transport convenience and re-assembly at site of work particularly in defence sector for high altitudes and far off situations.

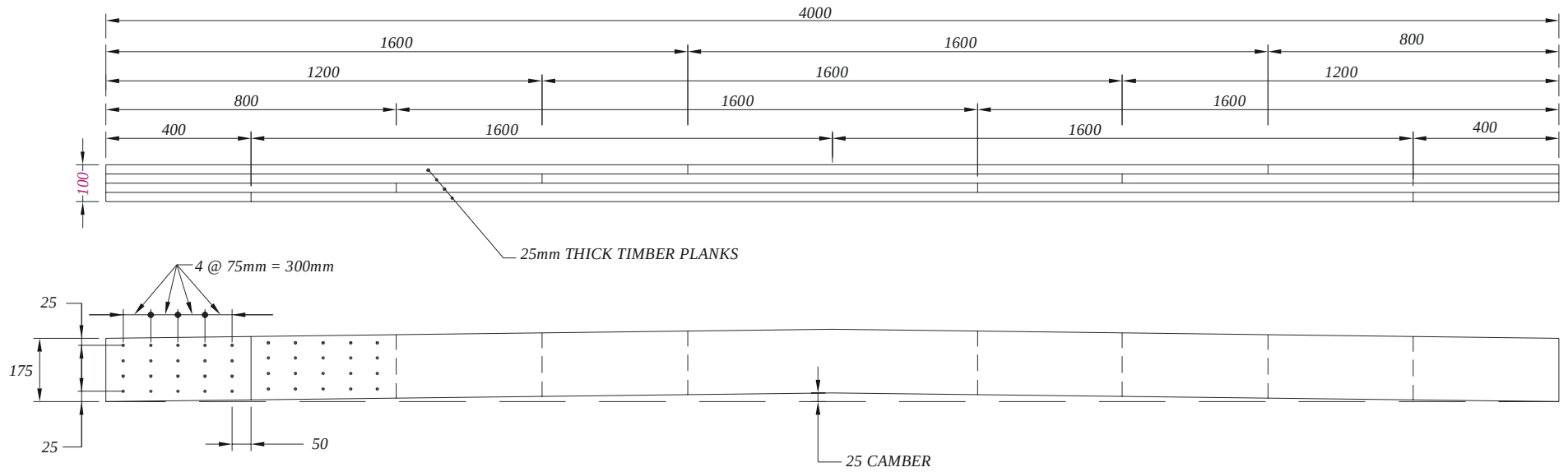
Designing is mainly influenced by the species, size of bolts, moisture conditions and the inclination of loadings to the grains. In principle bolted joints follow the pattern of rivetted joints in steel structures.

11.9.2 Design Considerations

- 11.9.2.1 Bolted timber construction shall be designed in accordance with 6. The concept of critical section, that is, the net section obtained by deducting the projected area of bolt-holes from the cross-sectional area of member is very important for the successful design and economy in timber.

11.9.2.2 Bolt Bearing Strength of Wood

The allowable load for a bolt in a joint consisting of two members (single shear) shall be taken as one half the allowable loads calculated for a three member joint (double shear) for the same $t'/d3$ ratio. The percentage of safe working compressive stress of timber on bolted joints for different $t'/d3$ ratios shall be as given in Table 11.9.1.



All dimensions in millimetres.

Fig. 11.8.1 Plan And Elevation Of A Typical Nailed Laminated Timber Beam

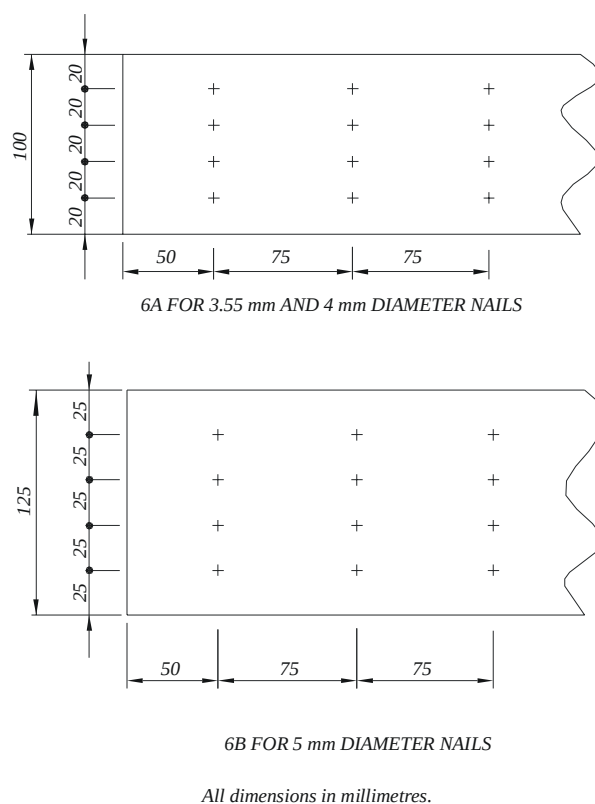


FIG. 11.8.2 Standard Lengthwise Spacing in Nailed Laminated Beam

11.9.2.2.1

Where a number of bolts are used in a joint, the allowable loads shall be the sum of the allowable loads for the individual bolts.

11.9.2.2.2

The factors for different bolt diameter used in calculating safe bearing stress perpendicular to grain in the joint shall be as given in Table 11.9.2.

11.9.2.3 Dimensions of Members

- The minimum thickness of the main member in mono-chord construction shall be 40 mm.
- The minimum thickness of side members shall be 20 mm and shall be half the thickness of main members.
- The minimum individual thickness of spaced member in split-chord construction shall be 20 mm and 25 mm for webs and chord members respectively.

11.9.2.4 Bolts and Bolting

- The diameter of bolt in the main member shall be so chosen to give larger slenderness (t'/d_3) ratio of bolt.
- There shall be more number of small diameter bolts rather than small number of large diameter bolts in a joint.
- A minimum of two bolts for nodal joints and four bolts for lengthening joints shall be provided.
- There shall be more number of rows rather than more bolts in a row.
- The bolt holes shall be of such diameter that the bolt can be driven easily.
- Washers shall be used between the head of bolt and wood surface as also between the nut and wood.

11.9.3 Arrangement of Bolts

11.9.3.1 The following spacing in bolted joints shall be followed (see Fig. 11.9.1):

- a) Spacing of Bolts in a Row — For parallel and perpendicular to grain loading = $4 d_3$
- b) Spacing Between Rows of Bolts

Table 11.9.1: Percentage of safe working compressive stress of timber for bolted joints in double shear

t'/d_3 ratio	Stress Percentage	
	Parallel to Grain	Perpendicular to Grain
	λ_1	λ_2
(1)	(2)	(3)
1.0	100	100
1.5	100	96
2.0	100	88
2.5	100	80
3.0	100	72
3.5	100	66
4.0	96	60
4.5	90	56
5.0	80	52
5.5	72	49
6.0	65	46
6.5	58	43
7.0	52	40
7.5	46	39
8.0	40	38
8.5	36	36
9.0	34	34
9.5	32	33
10.0	30	31
10.5	-	31
11.0	-	30
11.5	-	30
12.0	-	28

1) For perpendicular to grain loading — $2.5 d_3$ to $5 d_3$ ($2.5 d_3$ for t'/d_3 ratio of 2 and $5 d_3$ for t'/d_3 ratio of 6 or more. For ratios between 2 to 6 the spacing shall be obtained by interpolation.

2) For parallel to grain loading — At least $(N - 4) d_3$, with a minimum of $2.5 d_3$. Also governed by net area at critical section which should be 80 percent of the total area in bearing under all bolts.

- c) End Distance — $7d_3$ for soft woods in tension, $5 d_3$ for hardwoods in tension and $4 d_3$ for all species in compression.

- d) Edge Distance

- 1) For parallel to grain loading $1.5 d_3$ or half the distance between rows of bolts, whichever is greater.
- 2) For perpendicular to grain loading, (loaded edge distance) shall be at least $4 d_3$.

11.9.3.2 For inclined members, the spacing given above for perpendicular and parallel to grain of wood may be used as a guide and bolts arranged at the joint with respect to loading direction.

- 11.9.3.3 The bolts shall be arranged in such a manner so as to pass the centre of resistance of bolts through the inter-section of the gravity axis of the members.
- 11.9.3.4 Staggering of bolts shall be avoided as far as possible in case of members loaded parallel to grain of wood. For loads acting perpendicular to grain of wood, staggering is preferable to avoid splitting due to weather effects.

Table 11.9.2: Bolt diameter factor

Sl. No.	Diameter of Bolt (mm)	Diameter Factor (d _r)
(1)	(2)	(3)
i)	6	5.70
ii)	10	3.60
iii)	12	3.35
iv)	16	3.15
v)	20	3.05
vi)	22	3.00
vii)	25	2.90

11.9.3.5 Bolting

The bolt holes shall be bored or drilled perpendicular to the surface involved. Forcible driving of the bolts shall be avoided which may cause cracking or splitting of members. A bolt hole of 1.0 mm oversize may be used as a guide for preboring.

- 11.9.3.5.1 Bolts shall be tightened after one year of completion of structure and subsequently at an interval of two to three years.

11.9.4 Outline for Design of Bolted Joints

Allowable load on one bolt (unit bearing stress) in a joint with wooden splice plates shall not be greater than value of P, R, F as determined by one of the following equations:

- a) For Loads Parallel to Grain

$$P = f_{cp} a \lambda_1$$

- b) For Loads Perpendicular to Grain

$$R = f_{cp} a \lambda_2 d_f$$

- c) For Loads at an Angle to Grain

$$F = \frac{PR}{P \sin^2 \theta + R \cos^2 \theta}$$

11.10 DESIGN OF TIMBER CONNECTOR JOINTS**11.10.1**

In large span structures, the members have to transmit very heavy stresses requiring stronger jointing techniques with metallic rings or wooden disc-dowels. Improvised metallic ring connector is a split circular band of steel made from mild steel pipes. This is placed in the grooves cut into the contact faces of the timber members to be joined, the assembly being held together by means of a connecting bolt.

- 11.10.1.1 Dimensions of Members Variation of thickness of central (main) and side members affect the load carrying capacity of the joint.

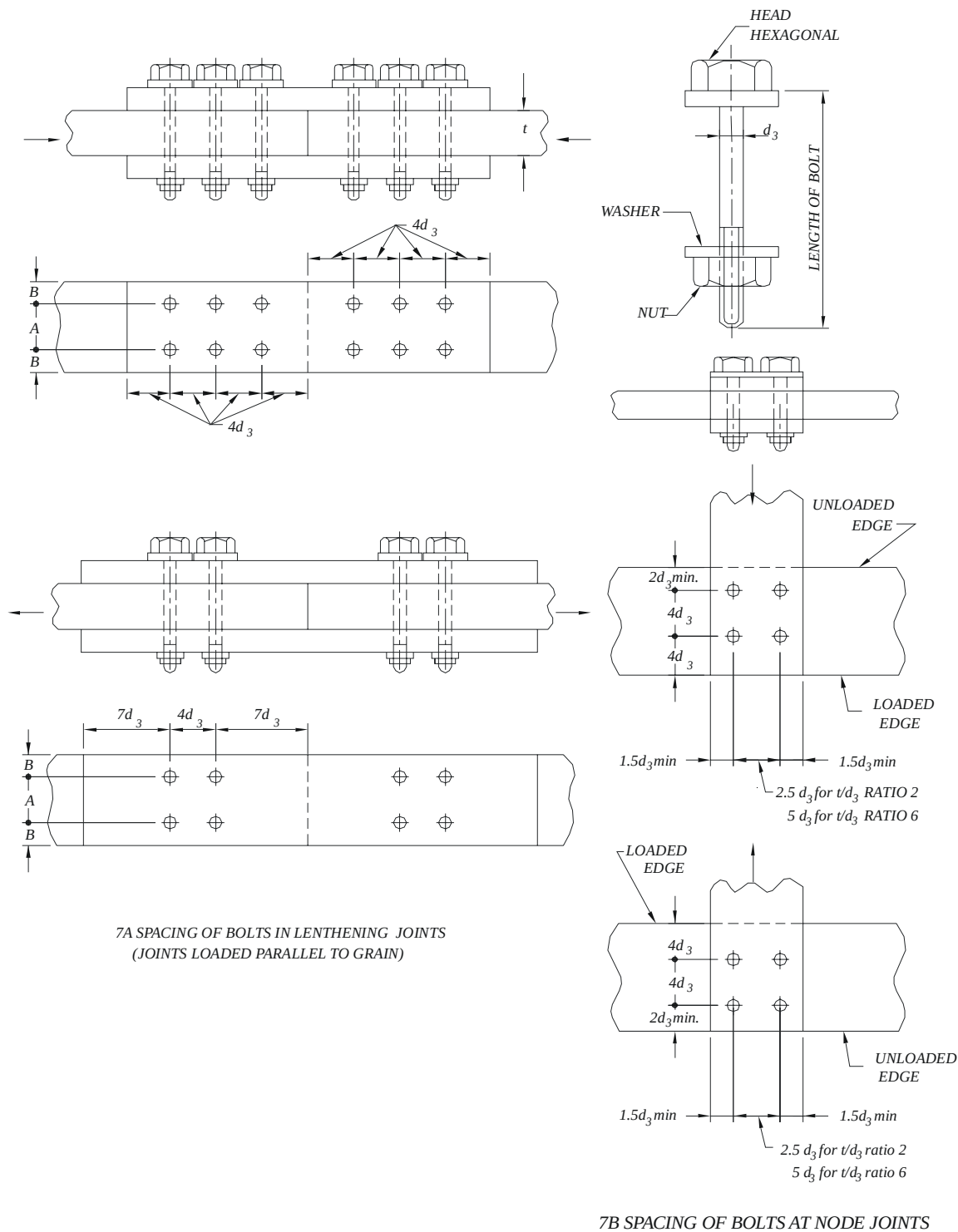


Fig. 11.9.1 Typical Spacing Of Bolts In Structural Joints

The thickness of main member shall be at least 57 mm and that of side member 38 mm with length and width of members governed by placement of connector at joint.

The metallic connector shall be so placed that the loaded edge distance is not less than the diameter of the connector and the end distance not less than 1.75 times the diameter on the loaded side.

11.10.1.2 Design Considerations

Figure 11.10.1 illustrates the primary stresses in a split ring connector joint under tension. The shaded areas represent the part of wood in shear, compression and tension. Related formulae for the same are indicated in Fig. 11.10.1.

For fabrication of structural members, a hole of the required size of the bolt is drilled into the member and a groove is made on the contact faces of the joint.

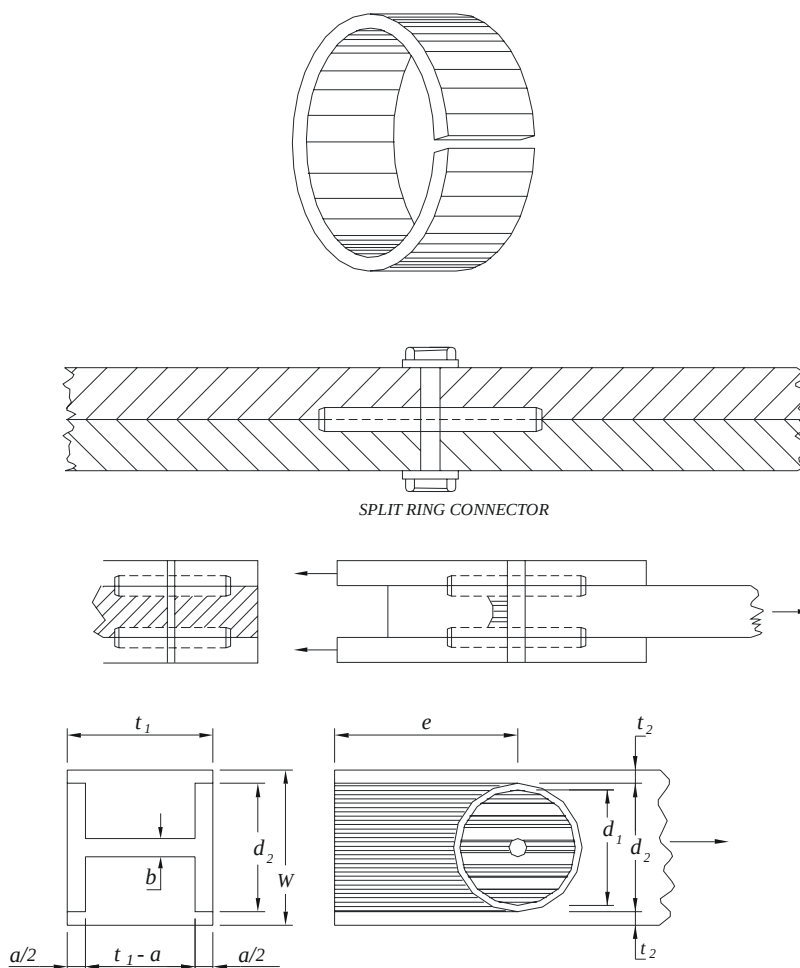


Fig. 11.10.1 Stress Distribution In A Split Ring Connector

11.10.2 Wooden Disc-Dowel

11.10.2.1 It is a circular hardwood disc generally tapered each way from the middle so as to form a double conical frustum. Such a disc is made to fit into preformed holes (recesses), half in one member and the other half in another, the assembly being held by one mild steel bolt through the centre of the disc to act as a coupling for keeping the jointed wooden members from spreading apart.

11.10.2.2 Dimensions of Members

The thickness of dowel may vary from 25 mm to 35 mm and diameter from 50 mm to 150 mm. The diameter of dowel shall be 3.25 to 3.50 times the thickness.

The edge clearance shall range from 12 mm to 20 mm as per the size of the dowel. The end clearance shall be at least equal to the diameter of dowel for joints subjected to tension and three-fourth the diameter for compression joints. Disc-dowel shall be turned from quarter sawn planks of seasoned material.

11.10.2.3 Choice of Species

Wood used for making dowels shall be fairly straight grained, free from excessive liability to shrink and warp, and retain shape well after seasoning species recommended include:

Babul

Sissoo

Pyinkado

Yon

NOTE — Data on the above species as per Table 11.4.1 except for the species Pyinkado, which is not an indigenous species.

11.10.2.4 Design Considerations

Figure 11.10.2 illustrates the forces on dowel in a lap joint and butt joint. Dowel is subjected to shearing at the mid-section, and compression along the grain at the bearing surfaces. For equal strength in both the forces, formula equations are given in Fig. 11.10.2 to determine the size of dowel.

The making of wooden discs may present some problems in the field, but they may be made in small workshop to the specifications of the designer. This is also economically important. Once the wood fittings are shop tailored and made, the construction process in the field is greatly simplified.

Theoretical safe loads in design shall be confirmed through sample tests.

11.11 GLUED LAMINATED CONSTRUCTION AND FINGER JOINTS

11.11.1 Developments in the field of synthetic adhesive have brought glueing techniques within the range of engineering practice. Timber members of larger cross-sections and long lengths can be fabricated from small sized planks by the process of gluelam. The term glued laminated timber construction as applied to structural members refers to various laminations glued together, either in straight or curved form, having grain of all laminations essentially parallel to the lengths of the member.

11.11.1.1 Choice of Glue

The adhesive used for glued laminated assembly are 'gap filling' type. A 'filler' in powder form is introduced in the adhesive. Structural adhesives are supplied either in powder form to which water is added or in resin form to which a hardener or catalyst is added. However, it is important that only boiling water proof (BWP) grade adhesives shall be used for fabrication of gluelam in tropical, high humid climates like India.

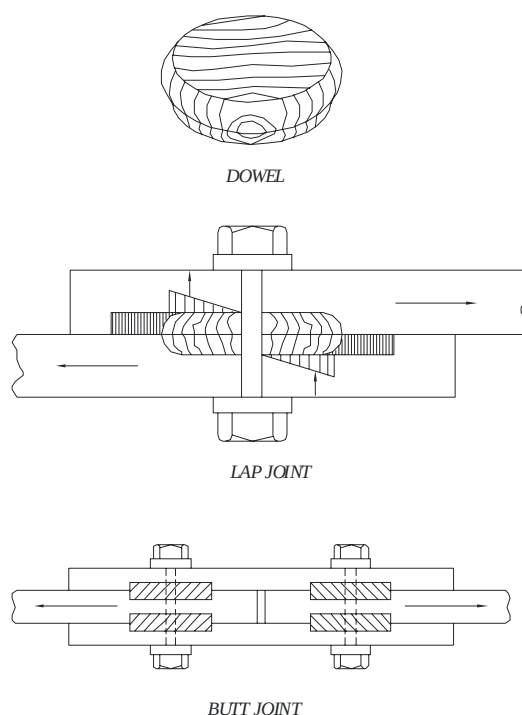
11.11.1.2 Manufacturing Schedule

In absence of a systematic flow-line in a factory, provisions of intermediate technology shall be created for manufacturing structural elements. The schedule involves steps:

- a) Drying of planks;
- b) Planning;
- c) End-jointing by scarfs or fingers;
- d) Machining of laminations;
- e) Setting up dry assembly of structural unit;
- f) Application of glue;
- g) Assembly and pressing the laminations;
- h) Curing the glue lines, as specified; and
- i) Finishing, protection and storage.

11.11.2

Finger joints are glued joints connecting timber members end-to-end (Fig. 11.11.1). Such joints shall be produced by cutting profiles (tapered projections) in the form of V-shaped grooves to the ends of timber planks or scantling to be joined, glueing the interfaces and then meeting the two ends together under pressure. Finger joints provide long lengths of timber, ideal for upgrading timber by permitting removal of defects, minimizing warping and reducing wastage by avoiding short off-cuts.



Lap Joint : Bolt in simple tension due to clockwise turning moment on dowel :

Butt Joint : No tilting moment in dowel due to balancing effect [dowels are in shear (no bending, shearing and tensile stress on bolts)]

Size of dowel for equal strength in both shearing and bearing.

$$\frac{\pi d^2}{4} \times S = d \times \frac{t}{2} \times c$$

Where

d = Mid diameter of the dowel

t = Thickness of dowel

s = Safe working stress in shear along grain, and

c = Safe compressive stress along grain.

NOTE- Symbols are exclusive in this figure.

Fig. 11.10.2 Distribution Of Forces In Dowel Joint

11.11.2.1 In finger joints the glued surfaces are on the side grain rather than on the end grain and the glue line is stressed in shear rather in tension.

11.11.2.1.1 The figures can be cut from edge-to-edge or from face-to-face. The difference is mainly in appearance, although bending strength increases if several fingers share the load. Thus a joist is slightly stronger with edge-to-edge finger joints and a plank is stronger with face-to-face finger joint.

11.11.2.1.2 For structural finger jointed members for interior dry locations, adhesives based on melamine formaldehyde cross linked polyvinyl acetate (PVA) are suited. For high humid and exterior conditions, phenol formaldehyde and resorcinol formaldehyde type adhesives are recommended. Proper adhesives should be selected in consultation with the designer and adhesive manufacturers.

11.11.2.2 Manufacturing Process

In the absence of sophisticated machinery, the finger joints shall be manufactured through intermediate technology with the following steps:

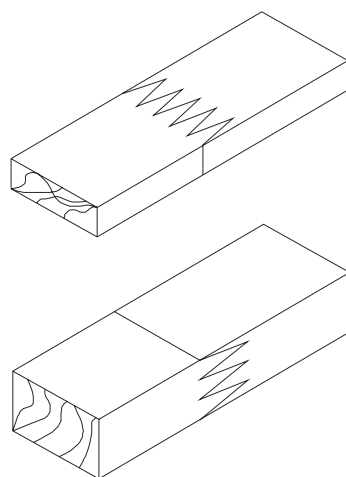
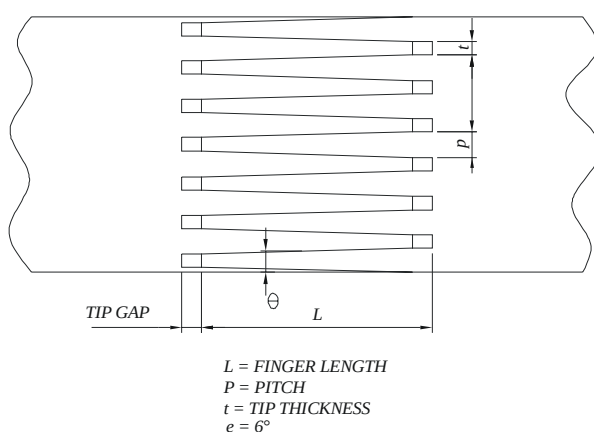
- a) Drying of wood,
- b) Removal of knots and other defects,
- c) Squaring the ends of the laminating planks,
- d) Cutting the profile of finger joint in the end grain,
- e) Applying adhesives on the finger interfaces,
- f) Pressing the joint together at specified pressure,
- g) Curing of adhesive line at specified temperature, and
- h) Planning of finger-jointed planks for smooth surface.

11.11.2.3 Strength

Strength of finger joints depends upon the geometry of the profile for structural purpose; this is generally 50 mm long, 12 mm pitch.

11.11.2.3.1 End joints shall be scattered in adjacent laminations, which shall not be located in very highly stressed outer laminations.

11.11.2.4 Tip thickness will be as small as practically possible.



ORIENTATION OF FINGER JOINTS

Fig. 11.11.1 Typical Finger Joint Geometry

11.12 LAMINATED VENEER LUMBER

11.12.1 Certain reconstituted lignocellulosic products with fibre oriented along a specific direction have been developed and are being adopted for load bearing applications. Laminated veneer lumber is one such product developed as a result of researches in plantation grown species of wood. Density of laminated veneer lumber ranges from 0.6 to 0.75.

11.12.1.1 Dimensions

Sizes of laminated veneer lumber composite shall be inclusive of margin for dressing and finishing unless manufactured to order. The margin for dressing and finishing shall not exceed 3mm in the width and thickness and 12mm in the length.

11.12.1.2 Permissible Defects

Jointing gaps — Not more than 3 mm wide, provided they are well staggered in their spacing and position between the successive plies.

Slope of grain — Not exceeding 1 in 10 in the face layers.

Tight knot — Three numbers up to 25 mm diameter in one square metre provided they are spaced 300 mm or more apart.

Warp — Not exceeding 1.5 mm per metre length.

11.12.1.3 Strength Requirements

The strength requirements for laminated veneer lumber shall be as per Table 11.12.1.

Table 11.12.1: Requirements of laminated veneer lumber

Sl. No. (1)	Properties (2)	Requirement (3)
i)	Modulus of rupture (N/mm ²), Min	50
ii)	Modulus of elasticity (N/mm ²), Min	7500
iii)	Compressive strength:	
	a) Parallel to grain (N/mm ²), Min	35
	b) Perpendicular to grain (N/mm ²), Min	50
iv)	Horizontal shear:	
	a) Parallel to laminac (N/mm ²), Min	6
	b) Perpendicular to laminac (N/mm ²), Min	8
v)	Tensile strength parallel to grain (N/mm ²), Min	55
vi)	Screw holding power:	
	a) Edge (N), Min	2300
	b) Face (N), Min	2700
vii)	Thickness swelling in 2 h water soaking (percent), Max	3

11.13 DESIGN OF GLUED LAMINATED BEAMS

11.13.1 General

Glued laminated structural members shall be fabricated only where there are adequate facilities for accurate sizing and surfacing of planks, uniform application of glue, prompt assembly, and application of adequate pressure and prescribed temperature for setting and curing of the glue. Design and fabrication shall be in accordance with established engineering principles and good practice. A glued laminated beam is a straight member made from a number of laminations assembled both ways either horizontally or vertically. While vertical

laminations have limitations in restricting the cross-section of a beam by width of the plank, horizontally laminated section offers wider scope to the designer in creating even the curved members.

Simple straight beams and joists are used for many structures from small domestic rafters or ridges to the light industrial structures.

11.13.2 Design

The design of glue laminated wood elements shall be in accordance with good engineering practice and shall take into consideration the species and grade of timber used, presence of defects, location of end joints in laminations, depth of beams and moisture contents expected while in service. Beams of large spans shall be designed with a suitable camber to assist in achieving the most cost effective section where deflection governs the design. The strength and stiffness of laminated beams is often governed by the quality of outer laminations. Glued laminated beams can be tapered to follow specific roof slopes across a building and/or to commensurate with the varying bending moments.

11.13.3 Material

Laminating boards shall not contain decay, knots or other strength reducing characteristics in excess of those sizes or amounts permitted by specifications. The moisture content shall approach that expected in service and shall in no case exceed 15 percent at the time of glueing. The moisture content of individual laminations in a structural member shall not differ by more than 3 percent at the time of glueing. Glue shall be of type suitable for the intended service of a structural member.

11.13.4 Fabrication/Manufacture

In order to assure a well-bonded and well-finished member of true shape and size, all equipments, end-Jointing, glue spread, assembly, pressing, curing or any other operation in connection with the manufacture of glued structural members shall be in accordance with the available good practices and as per glue manufacturers' instructions as applicable.

11.14 STRUCTURAL USE OF PLYWOOD

Unlike sawn timber, plywood is a layered panel product comprising veneers of wood bonded together with adjacent layers usually at right angles. As wood is strongest when stressed parallel to grain, and weak perpendicular to grain, the lay up or arrangement of veneers in the panel determines its properties. When the face grain of the plywood is parallel to the direction of stress, veneers parallel to the face grain carry almost all the load. Some information/guidelines for structural use of plywood are given in 11.14.1 to 11.14.3.

11.14.1 The plywood has a high strength to weight ratio, and is dimensionally stable material available in sheets of a number of thicknesses and construction. Plywood can be sawn, drilled and nailed with ordinary wood working tools. The glues used to bond these veneers together are derived from synthetic resins which are set and cured by heating. The properties of adhesives can determine the durability of plywood.

11.14.2 In glued plywood construction, structural plywood is glued to timber resulting in highly efficient and light structural components like web beams (I and box sections), (Fig. 11.15.1 and Fig. 11.15.2) stressed skin panels (Fig. 11.15.3) used for flooring and walling and pre-fabricated houses, cabins, etc.

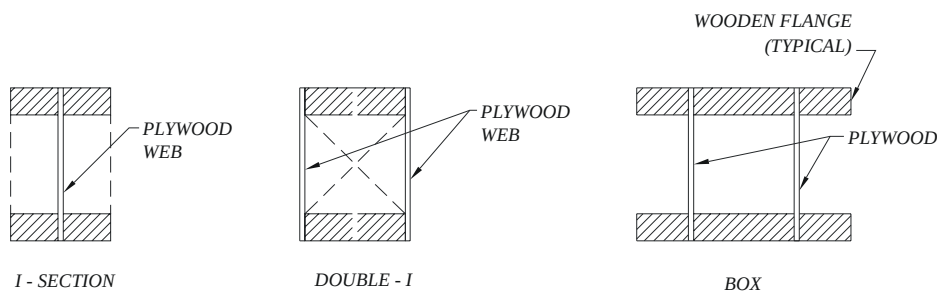


Fig. 11.15.1 Typical Cross - Section Of Web Beams

Glueing can be carried out by nail glueing techniques with special clamps. High shear strength of plywood in combination with high flexural strength and stiffness of wood result in structures characterized by high stiffness for even medium spans. Plywood can act as web transmitting shear stress in web bearing or stressed skin or sandwich construction. The effective moment of inertia of web beam and stressed skin construction depends on modular ratio that is, E of wood to E of plywood.

11.14.3 Structural plywood is also very efficient as cladding material in wood frame construction, such as houses. This type of sheathing is capable of resisting racking due to wind and quack forces. Structural plywood has been widely used as diaphragm (horizontal) as in roofing and flooring in timber frame construction. It has been established that 6 mm thick plywood can be used for sheathing and even for web and stressed skin construction, 9-12 mm thick plywood is suitable for beams, flooring diaphragms, etc. Phenol formaldehyde (PF) and PRF adhesive are suitable for fabrication of glued plywood components. 6 mm-12 mm thick structural plywood can be very well used as nailed or bolted gussets in fixing members of trusses or lattice girders or trussed rafters.

Normally, scarf joints are used for fixing plywood to required length and timber can be joined by using either finger or scarf joints. Arch panels, folded plates, shelves are other possibilities with this technique.

11.15 TRUSSED RAFTER

11.15.1 General

A roof truss is essentially a plane structure which is very stiff in the plane of the members, that is, the plane in which it is expected to carry loads, but very flexible in every other direction. Thus it can virtually be seen as a deep, narrow girder liable to buckling and twisting under loads. In order, therefore, to reduce this effect, eccentricity of loading and promote prefabrication for economy, low-pitched trussed rafters are designed with bolt ply/nail ply joints. Plywood as gussets, besides being simple have inherent constructional advantage of grain over solid wood for joints, and a better balance is achievable between the joint strength and the member strength.

Trussed rafters are light weight truss units spaced at close centres for limited spans to carry different types of roof loads. They are made from timber members of uniform thickness fastened together in one plane. The plywood gussets may be nailed or glued to the timber to form the joints. Conceptually a trussed rafter is a triangular pin jointed system, traditionally meant to carry the combined roof weight, cladding services and wind loads. There is considerable scope for saving timber by minimizing the sections through proper design without affecting structural and functional requirements.

Trussed rafters require to be supported only at their ends so that there is no need to provide load bearing internal walls, Purlins, etc are dispensed with and in comparison with traditional methods of construction they use less timber and considerably reduces of site labour, Mass production or reliable units can be carried out under workshop controls.

11.15.2 Design

Trussed rafter shall be designed to sustain the dead and imposed loads specified in Part 6 'Structural Design: Chapter 2 Loads on Buildings and Structures' and the combinations expected to occur. Extra stresses/ deflections during handling, transportation and erection shall be taken care of. Structural analysis, use of load-slip and moment, rotation characteristics of the individual joints may be used if feasible. Alternatively the maximum direct force in a member may be assessed to be given by an idealized pin-jointed framework, fully loaded with maximum dead and imposed load in the combination in which they may reasonably be expected to occur.

11.15.3 Timber

The species of timber including plantation grown species which can be used for trussed rafter construction and permissible stresses thereof shall be in accordance with Table 11.4.1. Moisture contents to be as per zonal requirements in accordance with 11.4.4.

11.15.4 Plywood

Boiling water resistant (BWR) grade preservative treated plywood shall be used. Introduction of a plywood gusset simplifies the jointing and in addition provides rigidity to the joint. Preservation of plywood and other panel

products shall be done in accordance with good practice prescribed by Bangladesh Forest Research Institute, Chittagong.

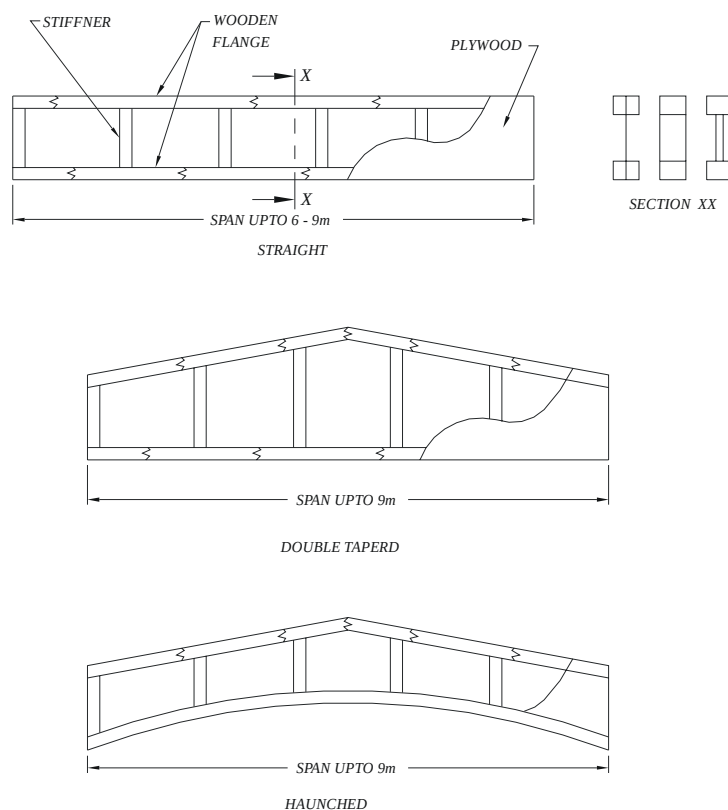


Fig. 11.15.2 Web Beam Configurations

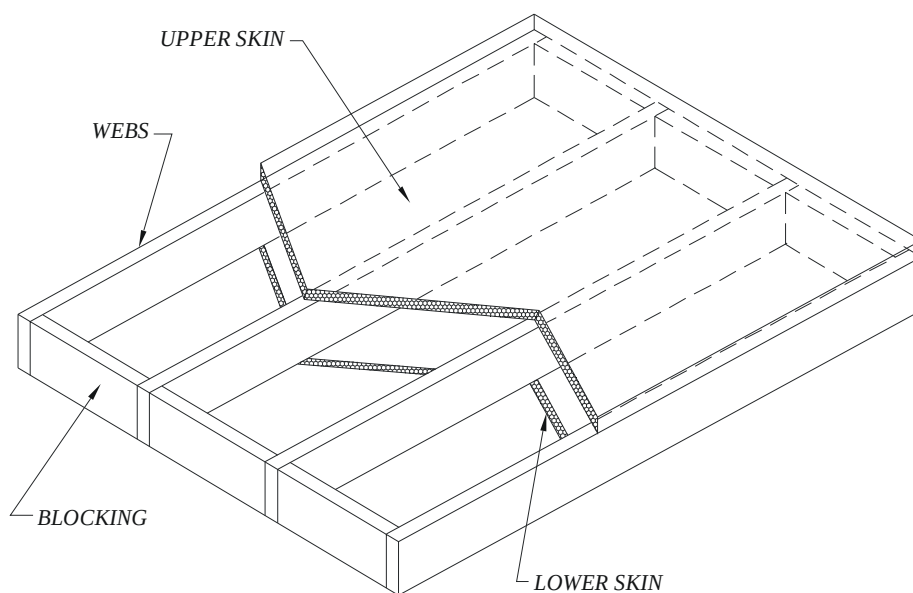


Fig. 11.15.3 Stressed Skin Panel Construction (Single Skin Or Double Skin)

11.16 STRUCTURAL SANDWICHES

11.16.1 General

Sandwich constructions are composites of different materials including wood based materials formed by bonding two thin facings of high strength material to a light weight core which provides a combination of desirable properties that are not attainable with the individual constituent materials (Fig. 11.16.1). The thin facings are usually of strong dense material since that are the principal load carrying members of the construction. The core must be stiff enough to ensure the faces remain at the correct distance apart. The sandwiches used as structural elements in building construction shall be adequately designed for their intended services and shall be fabricated only where there are adequate facilities for glueing or otherwise bonding cores to facings to ensure a strong and durable product. The entire assembly provides a structural element of high strength and stiffness in proportion to its mass.

Non-structural advantages can also be derived by proper selection of facing and core material for example, an impermeable facings can be used to serve as a moisture barrier for walls and roof panels and core may also be selected to provide thermal and/or acoustic insulation, fire resistance, etc, besides the dimensional stability.

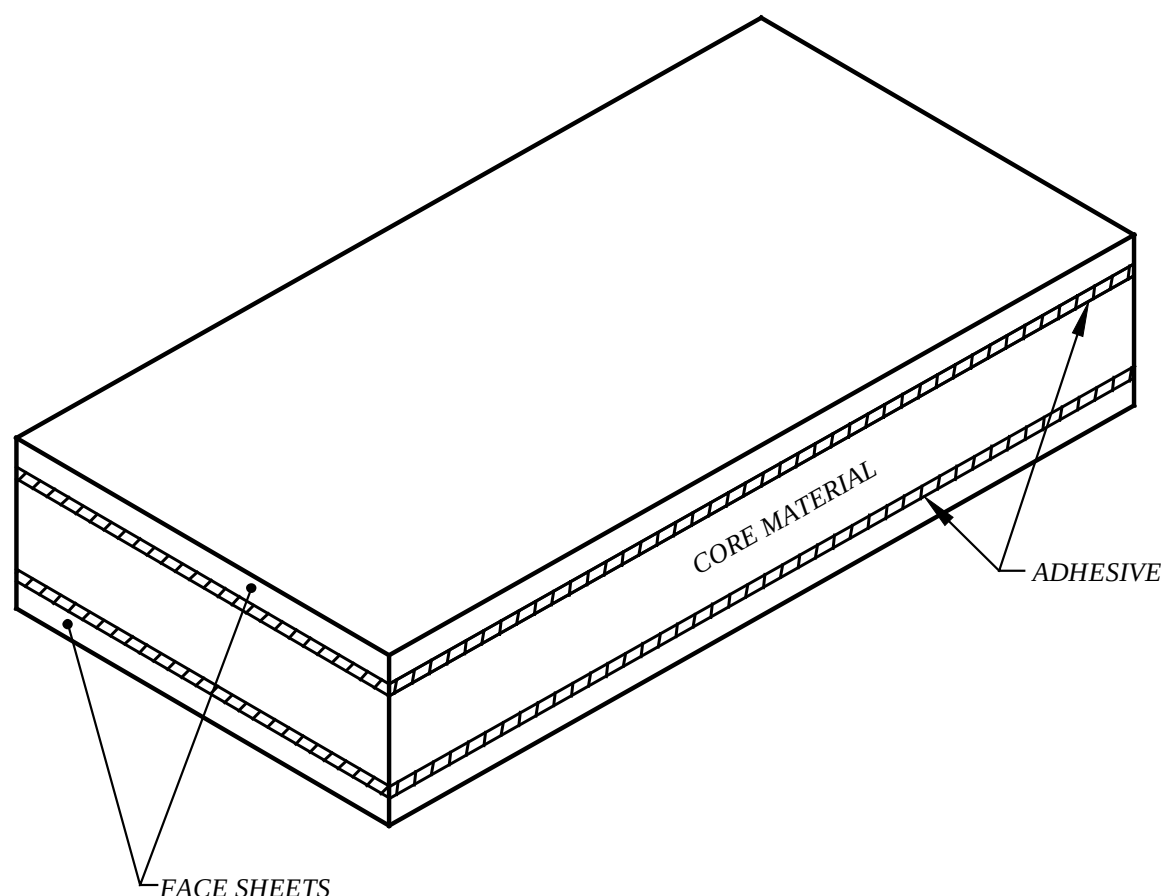


Fig. 11.16.1: Sandwich construction in structural applications

11.16.2 Cores

Sandwich cores shall be of such characteristics as to give to the required lateral support to the stressed facings to sustain or transmit the assumed loads or stresses. Core generally carries shearing loads and to support the thin facings due to compressive loads. Core shall maintain the strength and durability under the conditions of service for which their use is recommended. A material with low E and small shear modulus may be suitable.

11.16.3 Facings

Facings shall have sufficient strength and rigidity to resist stresses that may come upon them when fabricated into a sandwich construction. They shall be thick enough to carry compressive and tensile stresses and to resist puncture or denting that maybe expected in normal usages.

11.16.4 Designing

Structural designing may be comparable to the design of I-beams, the facings of the sandwich represent the flanges of the I-beam and the sandwich core I-beam web.

11.16.5 Tests

Tests shall include, as applicable, one or more of the following:

- a) Flexural strength/stiffness,
- b) Edge-wise compressions,
- c) Flat-wise compression,
- d) Shear in flat-wise plane,
- e) Flat-wise tensions,
- f) Flexural creep (creep behaviour of adhesive),
- g) Cantilever vibrations (dynamic property), and
- h) Weathering for dimensional stability.

11.17 LAMELLA ROOFING

11.17.1 General

The Lamella roofing offers an excellent architectural edifice in timber, amenable to prefabrication, light weight structure with high central clearance. It is essentially an arched structure formed by a system of intersecting skewed arches built-up of relatively short timber planks of uniform length and cross-section. Roof is designed as a two hinged arch with a depth equal to the depth of an individual lamella and width equal to the span of the building. The curved lamellas (planks) are bevelled and bored at the ends and bolted together at an angle, forming a network (grid) pattern of mutually braced and stiffened members (Fig. 11.17.1). The design shall be based on the balanced or unbalanced assumed load distribution used for roof arches. Effect of deformation or slip of joints under load on the induced stresses shall be considered in design. Thrust components in both transverse and longitudinal directions of the building due to skewness of the lamella arch shall be adequately resisted. Thrust at lamella joints shall be resisted by the moment of inertia in the continuous lamella and roof sheathing (decking) of lamella roofing. The interaction of arches in two directions adds to the strength and stability against horizontal forces. For design calculations several assumption tested and observed derivations, long-duration loading factors, seasoning advantages and effects of defects are taken into account.

11.17.2 Lamellas

Planking shall be of a grade of timber that is adequate in strength and stiffness to sustain the assumed loads, forces, thrust and bending moments generated in Lamella roofing. Lamella planks shall be seasoned to a moisture content approximating that they will attain in service. Lamella joints shall be proportioned so that allowable stresses at bearings of the non-continuous lamellas on the continuous lamellas or bearings under the head or washer of bolts are not exceeded.

11.17.3 Construction

Design and construction of lamella roofs in India assumes the roof surfaces to be cylindrical with every individual lamella an elliptic segment of an elliptical arch of constant curved length but of different curvature. Lamella construction is thus more of an art than science as there is no analytical method available for true generation of schedule of cutting lengths and curvature of curved members forming the lamella grid. Dependence of an engineer on the practical ingenuity of master carpenter is almost final. All the lamella joints shall be accurately cut and fitted to give full bearing without excessive deformation or slip. Bolts at lamella splices shall be adequate to hold the members in their proper position and shall not be over tightened to cause bending of the lamellas or mashing of wood under the bolt heads. Connection of lamellas to the end arches shall be adequate to transmit the thrust or any other force. Sufficient false work or sliding jig shall be provided for the support of lamella roof during actual construction/erection.

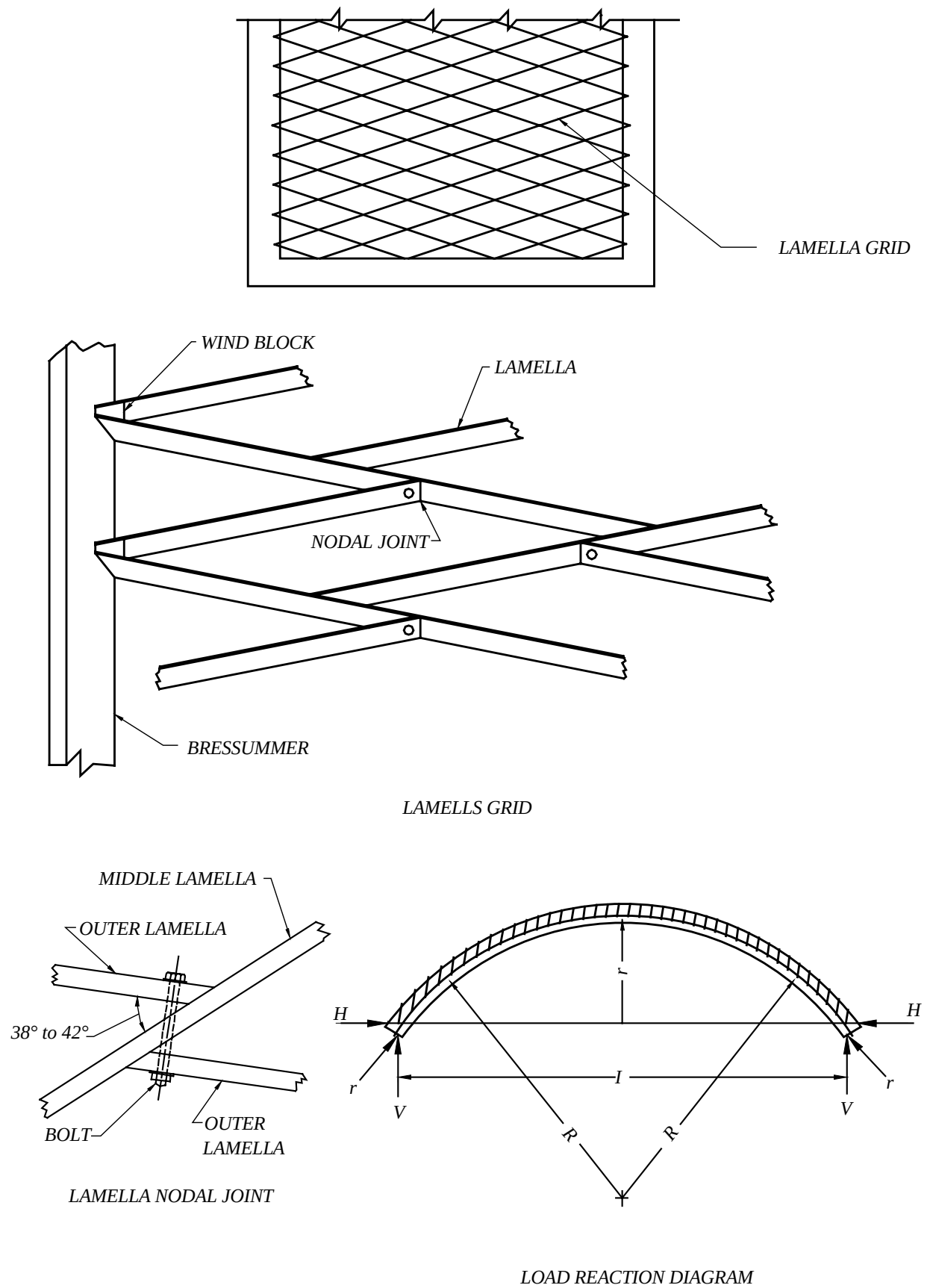


Fig. 11.17.1: Typical arrangement of Lamella roofing

11.18 NAIL AND SCREW HOLDING POWER OF TIMBER

11.18.1 General

One of the most common ways of joining timber pieces to one another is by means of common wire nails and wood screws. Timber is used for structural and nonstructural purposes in form of scantlings, rafters, joists, boarding, crating and packing cases, etc needing suitable methods of joining them. Nevertheless it is the timber which holds the nails or screws and as such pulling of the nails/screws is the chief factor which come into play predominantly. In structural nailed joints, nails are essentially loaded laterally, the design data for which is already available as standard code of practice. Data on holding power of nails/screws in different species is also useful for common commercial purposes. The resistance of mechanical fastenings is a function of the specific gravity of wood, direction of penetration with respect to the grain direction, depth of penetration and the diameter of fastener assuming that the spacing of fasteners should be adequate to preclude splitting of wood.

11.18.2 Nails

Nails are probably the most common and familiar fastener. They are of many types and sizes in accordance with the accepted standards [6-3 A(16)].

In general nails give stronger joints when driven into the side grain of wood than into the end grain. Nails perform best when loaded laterally as compared to axial withdrawal so the nailed joints should be designed for lateral nail bearing in structural design. Information on withdrawal resistance of nails is available and joints may be designed for that kind of loading as and when necessary.

11.18.3 Screw

Next to the hammer driven nails, the wood screw may be the most commonly used fastener. Wood screws are seldom used in structural work because of their primary advantage is in withdrawal resistance, for example, for fixing of ceiling "boards to joists, purlin cleats, besides the door hinges etc. They are of considerable structural importance in fixture design and manufacture. Wood screws are generally finished in a variety of head shapes and manufactured in various lengths for different screw diameters or gauges.

The withdrawal resistance of wood screws is a function of screw diameter, length of engagement of the threaded portion into the member, and the specific gravity of the species of wood. Withdrawal load capacity of wood screws are available for some species and joints may be designed accordingly. End grain load on wood screws are unreliable and wood screws shall not be used for that purpose.

11.19 PROTECTION AGAINST TERMITE ATTACK IN BUILDINGS

11.19.1 Two groups of organisms which affect the mechanical and aesthetic properties of wood in houses are fungi and insects. The most important wood destroying insects belong to termites and beetles. Of about 250 species of wood destroying termites recorded in India, not more than a dozen species attack building causing about 90 percent of the damage to timber and other cellulosic materials. Subterranean termites are the most destructive of the insects that infest wood in houses justifying prevention measures to be incorporated in the design and construction of buildings.

11.19.1.1 Control measures consist in isolating or sealing off the building from termites by chemical and non-chemical construction techniques. It is recognized that 95 percent damage is due to internal travel of the termites from ground upwards rather than external entry through entrance thus calling upon for appropriate control measures in accordance with good practices [6-3A(1 8)].

11.19.2 Chemical Methods

Termites live in soil in large colonies and damage the wooden structure in the buildings by eating up the wood or building nests in the wood. Poisoning the soil under and around the building is a normal recommended practice. Spraying of chemical solution in the trenches of foundations in and around walls, areas under floors before and after filling of earth, etc. In already constructed building the treatment can be given by digging trenches all around the building and then giving a liberal dose of chemicals and then closing the trenches.

11.19.3 Wood Preservatives

Natural resistance against organisms of quite a few wood species provides durability of timber without special protection measure. It is a property of heartwood while sapwood is normally always susceptible to attack by organisms. Preservatives should be well applied with sufficient penetration into timber. For engineers, architects and builders, the following are prime considerations for choice of preservatives:

- a) Inflammability of treated timber is not increased and mechanical properties are not decreased;
- b) Compatibility with the glue in laminated wood, plywood and board material;
- c) Water repellent effect is preferred;
- d) Possible suitability for priming coat;
- e) Possibility of painting and other finishes;
- f) Non-corrosive nature fasteners; and
- g) Influence on plastics, in case of metal rubber, tiles and concrete.

11.19.4 Constructional Method

Protection against potential problem of termite attack can simply be carried out by ordinary good construction which prevents a colony from gaining access by:

- a) periodic visual observations on termite galleries to be broken off;
- b) specially formed and properly installed metal shield at plinth level; and
- c) continuous floor slabs, apron floors and termite grooves on periphery of buildings.

Ferrocement Structures

12.1 SCOPE

This chapter covers selection, standards and testing of ferrocement materials, design criteria and approaches, construction methods, and maintenance and repair procedures of ferrocement structures. The provisions of this chapter are consistent with those of Chapter 6, except for the special requirements of ferrocement, such as reinforcement cover and limits on deflection.

12.2 TERMINOLOGY

12.2.1 Reinforcement Parameters

For the purpose of this chapter, the following parameters characterizing the reinforcement in ferrocement shall have the definitions given:

VOLUME FRACTION OF REINFORCEMENT (V_f): Volume fraction of reinforcement is the total volume of reinforcement divided by the volume of composite (reinforcement and matrix). For a composite reinforced with meshes with square openings, V_f shall be equally divided into V_{fl} and V_{ft} for the longitudinal and transverse directions, respectively. For other types of reinforcement, such as expanded metal, V_{fl} and V_{ft} may be unequal. Procedures for computation of V_f are shown in Appendix 6.12.A.

SPECIFIC SURFACE OF REINFORCEMENT (S_r): S_r is the total bonded area of reinforcement (interface area or area of the steel that comes in contact with the mortar) divided by the volume of the composite.

For a ferrocement plate of width b and depth h , the specific surface of reinforcement can be computed from

$$S_r = \frac{\Sigma o}{bh} \quad (12.2.1)$$

in which Σo is the total surface area of bonded reinforcement per unit length.

The relation between S_r and V_f when square grid wire meshes are used is

$$S_r = \frac{4V_f}{d_b} \quad (12.2.2)$$

where d_b is the diameter of the wire. For other types of reinforcement, such as expanded metal, S_{rl} and S_{rt} may be unequal.

EFFECTIVE MODULUS OF THE REINFORCEMENT: For welded steel meshes, effective modulus of the reinforcing system, E_r shall be taken equal to the elastic modulus of the steel wires. For other meshes, E_r shall be determined from tensile tests on the ferrocement composite as specified in Sec 12.8.

12.2.2 Notation

A_c	=	cross-sectional area of ferrocement composite
A_s	=	total effective cross-sectional area of reinforcement in the direction considered
$A_s = \sum_{i=1}^N A_{si}$		
A_{si}	=	effective cross-sectional area of reinforcement of mesh layer i in the direction considered
b	=	width of ferrocement section
c	=	distance from extreme compression fibre to neutral axis

C_c	=	resultant of the compressive stress block in ferrocement
C_{si}	=	compressive force in ferrocement layer i
d_c	=	clear cover of mortar over first layer of mesh
d_b	=	diameter or equivalent diameter of reinforcement used
d_i	=	distance from extreme compression fibre to centroid of reinforcing layer i
E_c	=	elastic modulus of mortar matrix
E_r	=	effective modulus of the reinforcing system
f'_c	=	specified compressive strength of ferrocement mortar
f_y	=	yield strength of mesh reinforcement or reinforcing bars
h	=	thickness of ferrocement section
M_n	=	nominal moment strength
N_n	=	nominal tensile strength
N	=	number of layers of mesh; nominal resistance
n_r	=	modular ratio of reinforcement
s	=	mesh opening or size
S_r	=	specific surface of reinforcement
S_{rl}	=	specific surface of reinforcement in the longitudinal direction
S_{rt}	=	specific surface of reinforcement in the transverse direction
T_{si}	=	tensile force in the ferrocement layer i
V_f	=	volume fraction of reinforcement
U	=	minimum required design strength
V_{fi}	=	volume fraction of reinforcement for mesh layer i
V_{fl}	=	volume fraction of reinforcement in the longitudinal direction
V_{ft}	=	volume fraction of reinforcement in the transverse direction
β_1	=	factor defining depth of rectangular stress block
η	=	global efficiency factor of embedded reinforcement in resisting tension or tensile bending loads
η_l	=	value of η when the load or stress is applied along the longitudinal direction of the mesh system or rod reinforcement
η_t	=	value of η when the load or stress is applied along the transverse direction of the mesh reinforcement system or rod reinforcement
η_θ	=	value of η when the load or stress is applied along a direction forming an angle with the longitudinal direction
ϵ_{si}	=	strain of mesh reinforcement at layer i
ϵ_y	=	nominal yield strain of mesh reinforcement = $\frac{f_y}{E_r}$
Σo	=	total surface area of bonded reinforcement per unit length
ϕ	=	strength reduction factor.

12.2.3 Definitions

ARMATURE: The total reinforcement system or skeletal reinforcement and mesh for a ferrocement element.

LONGITUDINAL DIRECTION: The roll direction (longer direction) of the mesh as produced in plant (see Fig 12.2.1).

SKELETAL REINFORCEMENT: A planar framework or widely spaced tied steel bars that provides shape and support for layers of mesh or fabric attached to either side.

SPRITZING: Spraying or squirting a mortar onto a surface.

TRANSVERSE DIRECTION: Direction of mesh normal to its longitudinal direction; also width direction of mesh as produced in plant (see Fig 12.2.1)

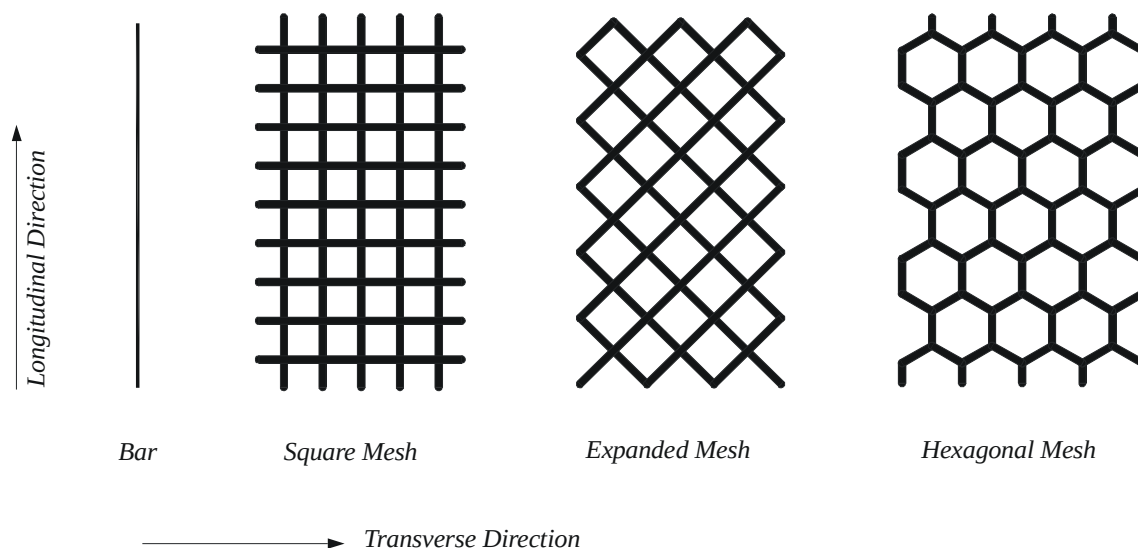


Fig. 12.2.1 Assumed Longitudinal and Transverse Directions of Reinforcement

12.3 MATERIALS

The material used in ferrocement consists primarily of mortar made with Portland cement, water and aggregate and the reinforcing mesh.

12.3.1 Cement

The cement shall comply with ASTM C150-85a, ASTM C595-85, or an equivalent standard. The cement shall be fresh, of uniform consistency, and free of lumps and foreign matter. It shall be stored under dry conditions for as short a duration as possible.

The choice of a particular cement shall depend on the service conditions. Service conditions can be classified as electrochemically passive or active. Land based structures such as ferrocement silos, bins, and water tanks can be considered as passive structures, except when in contact with sulphate bearing soils, in which case the use of sulphate resistant cement, such as ASTM Type II or Type V, may be necessary.

Blended hydraulic cement conforming to ASTM C595-85 Type 1 (PM), IS, 1 (SM), IS-A, IP, or IP-A can also be used.

Mineral admixtures, such as fly ash, silica fumes, or blast furnace slag, may be used to maintain a high volume fraction of fine filler material. When used, mineral admixtures shall comply with ASTM C618-85 and C989-85a. In addition to the possible improvement of flow ability, these materials also benefit long term strength gain, lower mortar permeability, and in some cases improved resistance to sulphates and chlorides.

12.3.2 Aggregates

Aggregate used in ferrocement shall be normal weight fine aggregate (sand). It shall comply with ASTM C33-86 requirements (for fine aggregate) or an equivalent standard. It shall be clean, inert, free of organic matter and deleterious substances, and relatively free of silt and clay.

The grading of fine aggregate shall be in accordance with the guidelines of Table 12.3.1. However, the maximum particle size shall be controlled by construction constraints such as mesh size and distance between layers. A maximum particle size passing sieve No. 16 (1.18 mm) may be considered appropriate in most applications. The sand shall be uniformly graded unless trial testing of mortar workability permits the use of a gap graded sand.

Aggregates that react with the alkalis in cement shall be avoided. When aggregates may be reactive, they shall be tested in accordance with ASTM C227-81. If proven reactive, the use of a pozzolan to suppress the reactivity shall be considered and evaluated in accordance with ASTM C441-81.

Table 12.3.1 Guidelines for Grading of Sand

Sieve Size	Per cent Passing by Weight
U.S. Standard Square Mesh	
No. 8 (2.36 mm)	80 - 100
No. 16 (1.18 mm)	50 - 85
No. 30 (0.60 mm)	25 - 60
No. 50 (0.30 mm)	10 - 30
No. 100 (0.15 mm)	2 - 10

12.3.3 Water

The mixing water shall be fresh, clean, and potable. The water shall be relatively free from organic matter, silt, oil, sugar, chloride, and acidic material. It shall have a pH ≥ 7 to minimize the reduction in pH of the mortar slurry. Salt water is not acceptable, but chlorinated drinking water can be used.

12.3.4 Admixtures

Conventional and high range water reducing admixtures (super plasticizers) shall conform to ASTM C494-86. Water reducing admixtures may be used to achieve an increase in sand content for the same design strength or a decrease in water content for the same workability. Decreases in water content result in lower shrinkage and less surface crazing. Retarders may be used in large time consuming plastering projects, especially in hot weather conditions.

If water tightness is important, such as in water or liquid retaining structures, special precautions shall be taken. To achieve water tightness, the water cement ratio shall preferably be kept below 0.4, crack widths limited (see Sec 12.4) and, if necessary, waterproofing coatings applied (see Sec 12.6.3).

Mineral admixtures such as fly ash (ASTM C618-85) can be added to the cement to increase workability and durability. Normally, 15 per cent of the cement can be replaced with mineral admixtures without appreciably reducing the strength. Pozzolanic admixtures may be added to replace part of the fine aggregates to improve plasticity. The tendency for some natural pozzolans to absorb water and thus adversely affect hydration of the cement phase shall be checked by measuring the water of absorption.

A quality matrix can be obtained without using any admixtures if experience has shown its applicability. Admixtures not covered in ASTM standards shall not be used.

12.3.5 Mix Proportioning

The ranges of mix proportions for common ferrocement applications shall be sand cement ratio by weight, 1.5 to 2.5, and water cement ratio by weight, 0.35 to 0.5. The higher the sand content, the higher the required water content to maintain the same workability. Fineness modulus of the sand, water cement ratio, and sand cement ratio shall be determined from trial batches to ensure a mix that can infiltrate (encapsulate) the mesh and develop a strong and dense matrix.

The moisture content of the aggregate shall be considered in the calculation of required water. Quantities of materials shall preferably be determined by weight.

The mix shall be as stiff as possible, provided it does not prevent full penetration of the mesh. Normally the slump of fresh mortar shall not exceed 50 mm. For most applications, the 28 day compressive strength of 75 by 150 mm moist cured cylinders shall not be less than 35 N/mm².

12.3.6 Reinforcement

The reinforcement shall be clean and free from deleterious materials such as dust, loose rust, coating of paint, oil, or similar substances.

Wire mesh with closely spaced wires is the most commonly used reinforcement in ferrocement. Expanded metal, welded wire fabric, wires or rods, prestressing tendons, and discontinuous fibers may also be used in special applications or for reasons of performance or economy.

12.4.1.2 In the design of ferrocement structures, members shall be proportioned for adequate strength in accordance with the provisions of this chapter using load factors and strength reduction factors specified in Chapter 6.

12.4.1.3 Ferrocement members may alternatively be designed using service loads and permissible service load stresses in accordance with the provisions of Sec 12.4.3.

12.4.1.4 All members shall also be designed to satisfy serviceability criteria in accordance with the provisions of Sec 12.4.4.

12.4.2 Strength Requirements

Ferrocement structures and structural members shall have a design strength at all sections at least equal to the required strengths for the factored load and load combinations stipulated in Chapter 1, General Design Requirements. Required strength U to resist dead load D and live load L shall be determined in accordance with Chapter 2, Sec 2.7.5.1.

Design strength provided by a member or cross-section in terms of axial load, bending moment, shear force, or stress shall be taken as the nominal strength calculated in accordance with requirements and assumptions of Chapter 6, Sec 6.1.4 multiplied by the strength reduction factor ϕ to satisfy the general relationship.

$$U \leq \phi N \quad (12.4.1)$$

where U is the factored load (equal to the minimum required design strength), N is the nominal resistance, and ϕ is a strength reduction factor defined in Sec 6.2.3.1.

Design strength for the mesh reinforcement shall be based on the yield strength f_y of the reinforcement but shall not exceed 690 N/mm^2 . Design yield strengths of various mesh reinforcement shall be in accordance with Table 12.4.1. These shall be used for design only when test data are not available. When tests for determination of yield strength are needed, they shall be conducted in accordance with Sec 12.8.2.3 and 12.8.2.4.

Table 12.4.1 Minimum Values of Yield Strength and Effective Modulus for Steel Meshes and Bars Recommended for Design

		Woven Square Mesh	Welded Square Mesh	Hexa- Gonal Mesh	Expanded Metal Mesh	Longitu- dinal Bars
Yield Strength	f_y N/mm^2	450	450	310	310	410
Effective Modulus	$(E_r)_{\text{long}}$ (N/mm^2)	138000	200000	104000	138000	200000
	$(E_r)_{\text{tran}}$ (N/mm^2)	165000	200000	69000	69000	-

12.4.2.1 Flexure:

The strain distribution at nominal moment resistance shall be assumed to be linear, and a rectangular stress block shall be used in computing the resultant compressive force acting on the concrete.

- a) Assumptions - Strength design of ferrocement members for flexure and axial loads shall be based on the following assumptions and on satisfaction of equilibrium and compatibility of strains.
 - i) Strain in reinforcement and mortar (concrete) shall be assumed directly proportional to the distance from the neutral axis.
 - ii) Maximum strain at extreme mortar (concrete) compression fibre shall be assumed equal to 0.003.
 - iii) Stress in reinforcement below specified yield strength f_y shall be taken as E_r times steel strain. For strains greater than that corresponding to f_y , stress in reinforcement shall be considered independent of strain and equal to f_y .

- iv) Tensile strength of mortar (concrete) shall be neglected in flexural strength calculations.
- v) Relationship between mortar (concrete) compressive stress distribution and mortar (concrete) strain may be considered satisfied by the use of the equivalent rectangular concrete stress distribution.
- b) Effective area of reinforcement - The area of reinforcement per layer of mesh considered effective to resist tensile stresses in a cracked ferrocement section shall be determined as follows:

$$A_{si} = \eta V_{fi} A_c \quad (12.4.2)$$

where

- A_{si} = effective area of reinforcement for mesh layer i
- η = global efficiency factor of mesh reinforcement in the loading direction considered
- V_{fi} = volume fraction of reinforcement for mesh layer i
- A_c = gross cross-sectional area of mortar (concrete) section.

The global efficiency factor η when multiplied by the volume fraction of reinforcement, gives the equivalent volume fraction (or equivalent reinforcement ratio) in the loading direction considered. In effect, it leads to an equivalent (effective) area of reinforcement per layer of mesh in that loading direction. For square meshes, $\eta = 0.5$ when loading is applied in one of the principal directions. For a reinforcing bar loaded along its axis, $\eta = 1.0$.

In the absence of values derived from tests for a particular mesh system, the values of η given in Table 12.4.2 for common types of mesh and loading direction may be used. The global efficiency factor shall apply whether the reinforcement is in the tension zone or in the compression zone.

The value of $\eta = 0.2$ for expanded metal mesh (Table 12.4.2) may not always be conservative, particularly in thicker sections in flexure with the mesh oriented in the short way diamond. The values in Table 12.4.2 shall be used for sections 50 mm or less in thickness, and tests conducted for global efficiency values for sections more than 50 mm in thickness.

Table 12.4.2 Recommended Design Values of the Global Efficiency Factor of Reinforcement for a Member in Uniaxial Tension or Bending

		Woven Square Mesh	Welded Square Mesh	Hexa- gonal Mesh	Expanded Metal Mesh	Longi- tudinal Bars
Global efficiency factor	Longitudinal, η_l	0.50	0.50	0.45	0.65	1
	Transverse η_t	0.50	0.50	0.30	0.20	0
	At 45° $\eta_{\theta=45}$	0.35	0.35	0.30	0.30	0.70

12.4.2.2 Tension:

The nominal resistance of cracked ferrocement elements subject to pure tensile loading shall be approximated by the load carrying capacity of the mesh reinforcement alone in the direction of loading by the following equation:

$$N_n = A_s f_y \quad (12.4.3)$$

where

- N_n = nominal tensile load resistance in direction considered
- A_s = effective cross-sectional area of reinforcement in direction considered
- f_y = yield stress of mesh reinforcement

The value of A_s is given by

$$A_s = \sum_{i=1}^N A_{si} \quad (12.4.4)$$

where

N = number of mesh layers

A_{si} = effective area of reinforcement for mesh layer i (Eq 12.4.2)

12.4.2.3 Compression:

As a first approximation, the nominal resistance of ferrocement sections subject to uniaxial compression shall be derived from the load carrying capacity of the unreinforced mortar (concrete) matrix assuming a uniform stress distribution of $0.85f'_c$ where f'_c is the design compressive strength of the mortar matrix. However, the transverse component of the reinforcement can contribute additional strength when square or rectangular wire meshes are used. Expanded mesh contributes virtually no strengthening beyond that achieved by the mortar alone. Slenderness effects of thin sections, which can reduce the load carrying capacity below that based on the design compressive strength shall be considered.

12.4.3 Service Load Design

12.4.3.1 Flexure:

For investigation of stresses at service loads, straight line theory (for flexure) shall be used with the following assumptions.

- Strains vary linearly with distance from the neutral axis.
- Stress strain relationships of mortar (concrete) and reinforcement are linear for stresses less than or equal to permissible service load stresses.
- Mortar (concrete) resists no tension.
- Perfect bond exists between steel and mortar (concrete).

To compute stresses and strains for a given loading, the cracked transformed section shall be used. The effective area of each layer of mesh reinforcement shall be determined from Eq (12.4.2). The same value of modular ratio $n_r = E_r/E_c$ may be used for both tensile and compressive reinforcement. Recommended design values of E_r are given in Table 12.4.1. Once the neutral axis is determined, the analysis shall proceed as for reinforced concrete beams or columns having several layers of steel and subject to pure bending.

12.4.3.2 Allowable Tensile Stress:

The allowable tensile stress in the mesh reinforcement under service conditions shall be taken as $0.60 f_y$ where f_y is the yield strength. For liquid retaining and sanitary structures, the allowable tensile stress shall be limited to 200 N/mm^2 . Consideration shall be given to increase the allowable tensile stresses if crack width measurements on a model test indicate that a higher stress will not impair performance.

12.4.3.3 Allowable Compressive Stress:

The allowable compressive stress in either the mortar (concrete) or the ferrocement composite shall be taken as where f'_c is the specified compressive strength of the mortar. Measurements of the mortar compressive strength shall be obtained from tests on 75 mm x 150 mm cylinders.

12.4.4 Serviceability Requirements

Ferrocement structures shall generally satisfy the intent of the serviceability requirements of Chapter 6 except for the concrete cover.

12.4.4.1 Crack Width Limitations:

The maximum value of crack width under service load conditions shall be less than 0.10 mm for noncorrosive environments and 0.05 mm for corrosive environments and/or water retaining structures.

12.4.4.2 Fatigue Stress Range:

For ferrocement structures to sustain a minimum fatigue life of two million cycles, the stress range in the reinforcement shall be limited to 200 N/mm^2 . A stress range of 350 N/mm^2 shall be used for one million cycles. Higher values may be considered if justified by tests.

12.4.4.3 Corrosion Durability:

Particular care shall be taken to ensure a durable mortar matrix and optimize the parameters that reduce the risk of corrosion.

12.4.4.4 Deflection Limitation:

Since ferrocement in thin sections is very flexible and its design is very likely to be controlled by criteria other than deflection, no particular deflection limitation is recommended.

12.4.5 Particular Design Parameters

12.4.5.1 The cover of the reinforcement shall be about twice the diameter of the mesh wire or thickness of other reinforcement used. A smaller cover is acceptable provided the reinforcement is not susceptible to rapid corrosion, the surface is protected by an appropriate coating, and the crack width is limited to 0.05 mm. For ferrocement elements of thickness less than 25 mm, a cover of the order of 2 mm shall be provided.

12.4.5.2 For a given ferrocement cross-section of total thickness h , the mesh opening shall not be larger than h .

12.4.5.3 For nonprestressed water retaining structures the total volume fraction of reinforcement shall not be less than 3.5 per cent and the total specific surface of reinforcement shall not be less than $0.16 \text{ mm}^2/\text{mm}^3$.

12.4.5.4 In computing the specific surface of the reinforcement, the contribution of fibres added to the matrix shall be considered, while the fibre contribution may be ignored in computing the volume fraction of reinforcement.

12.4.5.5 If skeletal reinforcement is used, the skeletal reinforcement shall not occupy more than 50 per cent of the thickness of the ferrocement composite.

12.4.5.6 For a given volume fraction of reinforcement, better performance in terms of crack widths, water tightness, and ductility may be achieved by uniform distribution of the reinforcement throughout the thickness and by increasing its specific surface. A minimum of two layers of mesh shall be acceptable, but more than two layers of reinforcement are preferable.

12.4.6 Design Aids

The following nondimensional equation can be used to predict the nominal moment strength of ferrocement beams subjected to pure bending:

$$\frac{M_n}{f'_c b h^2 \eta} = 0.005 + 0.422 \left(V_f f_y / f'_c \right) - 0.0772 \left(V_f f_y / f'_c \right)^2 \quad (12.4.5)$$

A design graph representing Eq (12.4.5) is given in Fig 12.4.2.

12.5 FABRICATION**12.5.1 General Requirements**

This section specifies the requirements for the mixing, placing, and handling of materials used in ferrocement construction.

12.5.1.1 Planning:

Plastering for fabrication with ferrocement shall be continuous through the completion of the job. This requires a large number of workers involved in plastering and in maintaining a constant supply of materials during work,

most often in confined work spaces. Adequate bond at cold joints may be achieved through surface roughness or treatment with bonding agents. Retarders may be useful in large time consuming plastering projects, especially in hot weather conditions. Planning for the job shall take into account all these requirements.

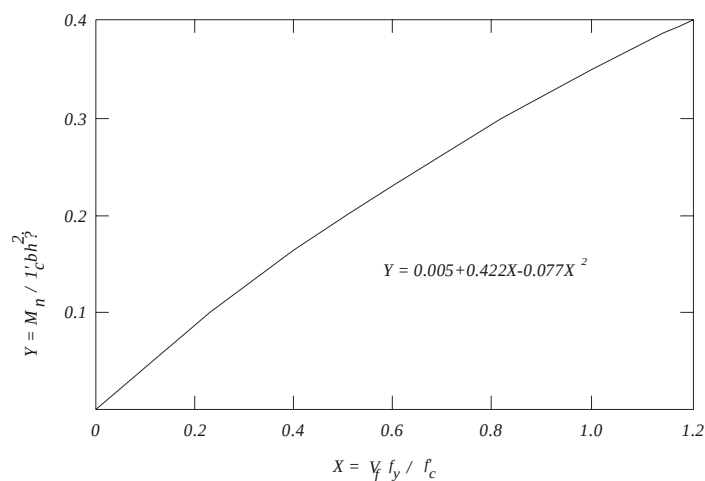


Fig. 12.4.2 Chart for Strength Design of Ferrocement in Bending

12.5.1.2 Mixing:

Any method, including hand mixing, which assures a homogeneous mixture of ingredients shall be satisfactory. Mixing may be accomplished in a mortar mixer with a spiral blade or paddles inside a stationary drum or in a pan type mixer. The use of rotating drum mixers with fins affixed to the sides shall not be permitted. Mix ingredients shall be carefully batched by weight, including the water, and added or charged in the mixer so that there is no caking. Mix water shall be accurately weighed so that the water cement ratio is controlled. The water cement ratio shall be as low as possible but the sand cement ratio shall be adjusted to provide a fluid mix for initial penetration of the armature followed by a stiffer more heavily sanded mix at the finish. Mortar shall be mixed in batches so that mortar is plastered within an hour after mixing. Retempering of the mortar shall be prohibited.

12.5.1.3 Mortar Placement:

Mortar shall generally be placed by hand plastering. In this process, the mortar is forced through the mesh. Alternatively, the mortar may be shot through a spray gun device.

12.5.1.4 Finishing:

Surfaces shall be finished to assure proper cover to the last mesh layer. The surface finish shall be slightly roughened if a surface coating is to be bonded later.

Surfaces that are too smooth shall be mechanically abraded by sandblasting or other means of mechanical abrasion. Alternatively, such surfaces may be etched with phosphoric acid, provided the residue left by it will not interfere with specified finishes. Mild solutions of muriatic acid may be applied with proper attention to corrosion potential. Additional care shall be taken when plastering around openings.

12.5.1.5 Curing:

Moist or wet curing is essential for ferrocement concrete construction. The low water cement ratio and high cement factors create a demand for large quantities of free water in the hydration process, and the amount permitted to evaporate into the air shall be kept to an absolute minimum. The use of fogging devices under a moisture retaining enclosure is desirable. A double layer of soaked burlap covered with polyethylene or a soaker hose may also be used. Continuous wetting of the surface or of wet burlap or the like shall be maintained to avoid dry spots. Curing shall start within a reasonable time after application of the finishing layer.

12.5.2 Construction Methods

All methods shall have high level quality control criteria to achieve the complete encapsulation of several layers of reinforcing mesh by a well compacted mortar or concrete matrix with a minimum of entrapped air. The most

appropriate fabrication technique shall be decided on the basis of the nature of the particular ferrocement application, the availability of mixing, handling, and placing machinery, and the skill and cost of available labour. Several recommended construction methods are outlined in the following subsections.

12.5.2.1 Armature System :

The armature system is a framework of tied reinforcing bars (skeletal steel) to which layers of reinforcing mesh are attached on each side. Mortar is then applied from one side and forced through the mesh layers towards the other side, as shown in Fig 12.5.1.

The skeletal steel can assume any shape. Diameter of the steel bars depends on the size of the structure. Skeletal steel shall be cut to specified lengths, bent to the proper profile, and tied in proper sequence. Sufficient embedment lengths shall be provided to ensure continuity. For bar sizes 6 mm or less, lap lengths from 230 to 300 mm may be sufficient. The required number of layers of mesh shall be tied to each side of the skeletal steel frame.

12.5.2.2 Closed-mould System :

The mortar is applied from one side through several layers of mesh or mesh and rod combinations that have been stapled or otherwise held in position against the surface of a closed mould, i.e. a male mould or a female mould. The mould may remain as a permanent part of the finished ferrocement structure. If removed, treatment with release agents may be needed. The use of the closed mould system represented in Fig 12.5.2 tends to eliminate the use of rods or bars, thus permitting an essentially all mesh reinforcement. It requires that plastering be done from one side only.

12.5.2.3 Integral-mould System:

An integral mould is first constructed by application of mortar from one or two sides onto a semi-rigid framework made with a minimum number of mesh layers. This forms, after mortar setting, a rigid but low quality ferrocement mould onto which further layer of reinforcing mesh and mortar shall be applied on both sides. Alternatively, the integral mould may be formed using rigid insulation materials, such as polystyrene or polyurethane, as the core. A schematic description of this system is given in Fig 12.5.3.

12.5.2.4 Open-mould System:

In the open-mould system, mortar is applied from one side through layers of mesh or mesh and rods attached to an open mould made of a lattice of wood strips. The form, shown in Fig 12.5.4, is coated with a release agent or entirely covered with polyethylene sheeting (thereby forming a closed but nonrigid and transparent mould) to facilitate mould removal and to permit observation and/or repair during the mortar application process.

This system is similar to the closed-mould system in which the mortar is applied from one side, at least until the mould can be removed. It enables at least part of the underside of the mould to be viewed and repaired, where necessary, to ensure complete and thorough impregnation of the mesh.

12.6 MAINTENANCE

12.6.1 General

Terrestrial structures are susceptible to deterioration from pollutants in ground water and those that precipitate from the air (acid rain). Environmental temperature and humidity variations also affect ferrocement durability and maintenance procedures.

Maintenance shall involve detecting and filling voids, replacing spalled cover, providing protective coatings, and cosmetic treatment of surface blemishes. Due to the thin cover in ferrocement, muriatic acid (hydrochloric acid) shall be used with extreme caution. Phosphoric acid and other nonchloride cleaners shall be the specified alternative (see Sec 12.5.1.4).

Repairs not involving large quantities of materials shall be accomplished by hand. Emphasis shall be placed on the ability of the repair material to penetrate the mesh cage, to fully coat the reinforcing to inhibit corrosion, and to bond to the substrate. Rapid set and strength gain shall be the overriding considerations for emergency repairs. Protective coatings shall bond well and be alkali tolerant, thermally compatible, and resistant to environmental pollutants and ultraviolet radiation, if exposed.

12.6.2 Blemish and Stain Removal

12.6.2.1 General:

Since ferrocement is usually less porous than conventional concrete, stains do not penetrate very deep in the mortar matrix. Care shall be taken when preparing the surface not to diminish the thin cover of mortar over ferrocement reinforcement.

12.6.2.2 Construction Blemishes:

Construction blemishes are often caused by improper selection or use of materials, faulty workmanship, uneven evaporation, and uneven curing. Care shall be exercised to minimize these and the following causes of blemishes in ferrocement.

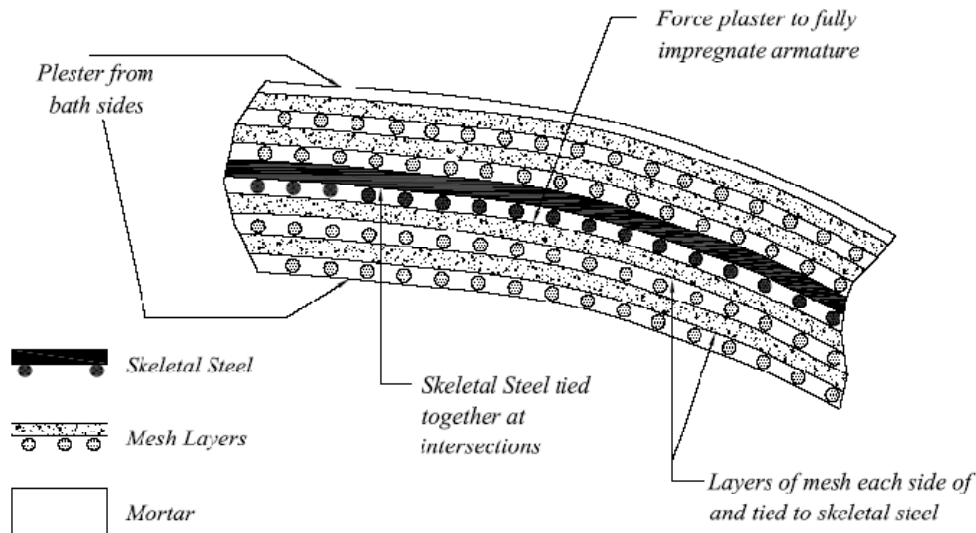


Fig. 12.5.1 Armature System

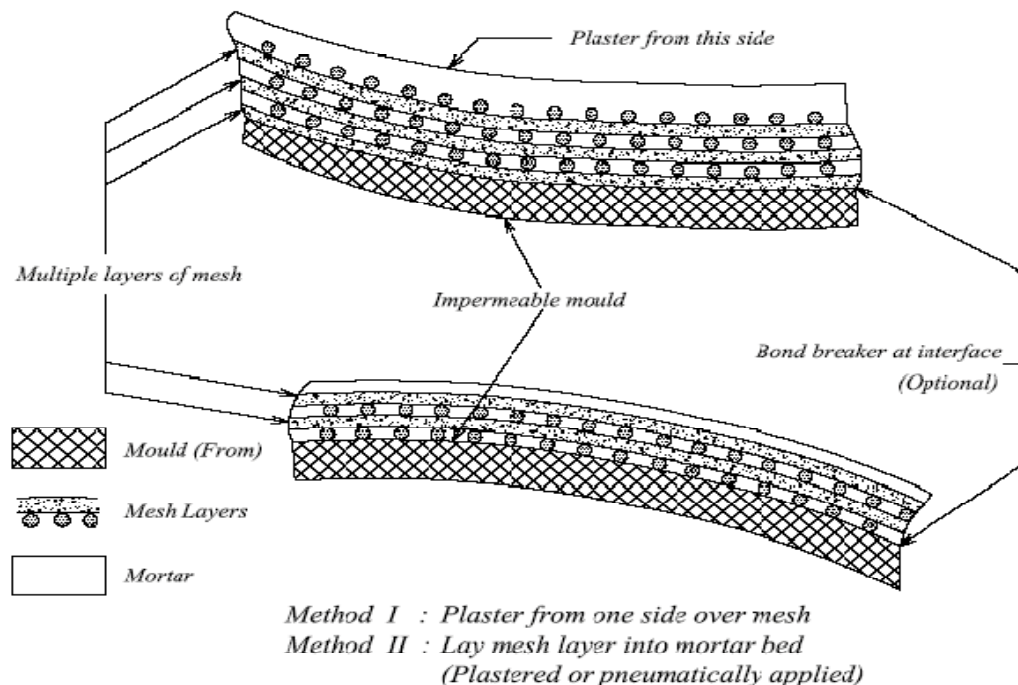


Fig. 12.5.2 Closed-mould System

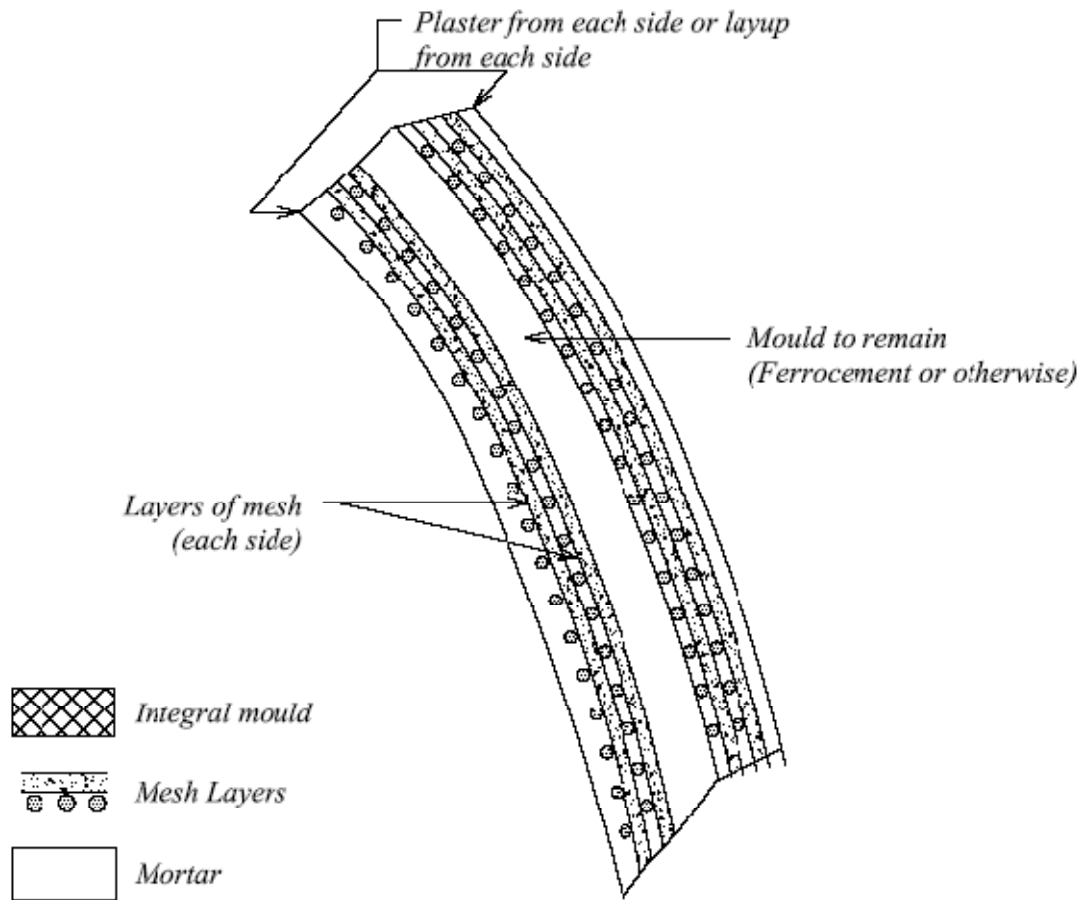


Fig. 12.5.3 Integral-mould System

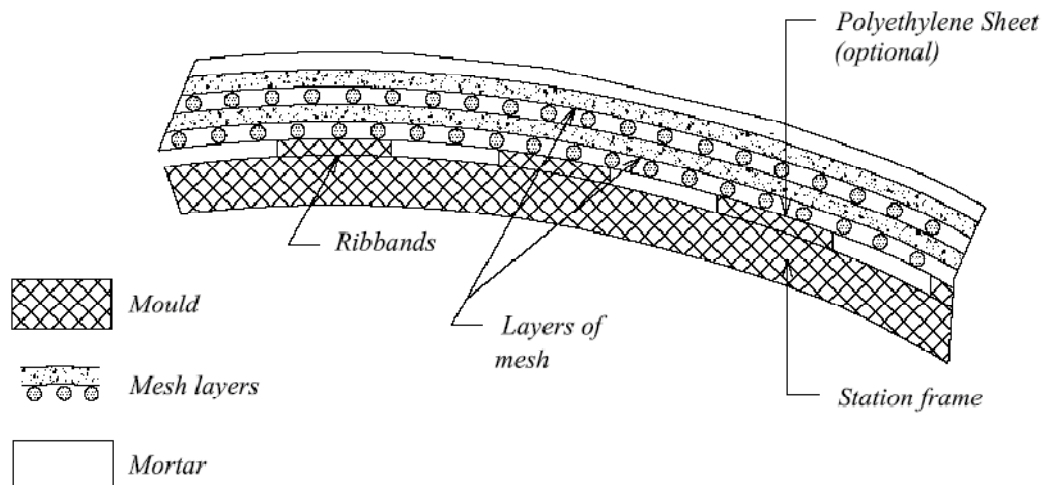


Fig. 12.5.4 Open-mould System

- Cement from different mills will cause colour variation, although most of the colour in mortar is due to the sand component. Where appearance is critical, care shall be taken to obtain sand from a single source and have it thoroughly washed.
- Mottling results from the use of calcium chloride or high alkali cement combined with uneven curing.
- The use of polyethylene sheet material to cover surfaces promotes uneven curing.

- d) The water cement ratio affects tone and surface appearance. Low water cement ratio will result in a darker appearance.
- e) Hard steel toweling densifies the surface, causing more rapid drying and also leaving a darkened surface.

12.6.2.3 Stain Removal:

Treatment of stains shall be done promptly after discoloration appears. Thorough flushing and brushing with a stiff bristle brush and detergent is the first approach. If this is ineffective, a dilute (about three per cent) solution of phosphoric or acetic acid shall be applied. Another chemical treatment that may be considered safe and effective is a 20 to 30 per cent solution of di-ammonium citrate, a mild acid that attacks calcium carbonates and calcium hydroxides. This treatment makes the surface more porous and promotes hydration.

When a stain has penetrated too deeply to be removed by surface chemical application and scrubbing, a poultice or a bandage may be needed. A poultice is intended to dissolve the stain and absorb it into the poultice. The poultice is made by mixing one or more chemicals such as a solution of phosphoric acid with a fine inert power such as talc, whiting, hydrated lime, or diatomaceous earth to form a paste. The paste is spread in a thick layer over the stain and allowed to dry. A bandage may consist of a few layers of cloth or paper toweling soaked in a chemical solution. More than one application of a poultice or bandage may be needed for stubborn stains.

Caution:

Most of the chemicals used to remove stains are toxic and require safeguards against skin contact and inhalation. Whenever acids are used, surfaces shall first be saturated with water or the dissolved stain material may migrate deeper into the concrete and reappear at a later date as efflorescence.

12.6.2.4 Efflorescence:

Efflorescence is caused by deposition of salts on the surface due to the evaporation of migrating water bearing salts from within ferrocement; it is typically associated with a porous ferrocement. Water cement ratio shall be limited to within 0.4 and the mortar well compacted to minimize efflorescence. Voids, if present, may be treated by breaking into with a hammer and replastering. Alternatively, voids may be drilled into with a masonry bit and repaired by injecting a non shrinking cement grout.

12.6.3 Protective Surface Treatments

12.6.3.1 General:

Good quality mortar has excellent resistance to weathering. The application of protective surface treatments can improve the performance of ferrocement and extend its useful service life. Surface treatments shall be used to improve appearance, harden the surface, and reduce permeability, thus guarding against the corrosive action of acids, alkaline salts, and organic substances.

12.6.3.2 Hardeners:

Hardeners may be used to protect the ferrocement surface or to seal and prepare it for application of paints. When a sodium silicate hardener is used, it shall be diluted with water. The actual proportion of water to be used shall depend on the manufacturer's recommendation. The hardener shall be applied in multiple coats with the first coat being more dilute than the subsequent ones. Each coat must be completely dry before the next coat is applied.

Other hardeners that seal and prepare the surface for application of oil base paints are magnesium fluorosilicate and zinc fluorosilicate. The treatment shall consist of two or more applications. A solution containing about 1 kg of fluorosilicate crystals per 10 litres of water shall be used for the first application; and a solution containing 2.4 kg per 10 litres of water shall be used for subsequent applications. After the last application has dried, the surface shall be brushed and washed with water to remove any crystals that may have formed.

12.6.3.3 Coatings :

When resistance to abrasion is desired, ferrocement surfaces may be coated with polyurethanes, especially those furnished in two part mixtures. Coatings formulated from acrylics may be used to provide resistance to sunlight and weathering. Water based acrylic latex house paints may be used for application to damp surfaces.

For any surface opposite a surface sealed with an impermeable coating, an acrylic coating formulated to allow the escape of water vapour shall be specified.

12.7 DAMAGE REPAIR

12.7.1 Common Types of Damage

Ferrocement structures shall be inspected, as part of a regular maintenance programme, to detect any of the following types of damage. Appropriate repair measures shall then be taken.

12.7.1.1 Delaminations :

Delaminations occur when ferrocement splits between layers in laminated constructions due to springing back or bridging of the mesh during construction. Delamination sometimes occurs at or near the neutral axis under impact or flexure when there are many voids in the interior layers. Such areas give off a hollow sound when tapped with a hammer or stroked with a steel bar.

12.7.1.2 Spalls:

A spall is defined as a depression resulting when a fragment is detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the mass. Spalls shall be considered large when their size exceeds approximately 20 mm in depth or 150 mm in any dimension, and shall be repaired by replastering.

Spalls are usually caused by corrosion of steel, which causes an expansive pressure within the ferrocement. Chlorides in the concrete greatly increase the potential for corrosion of the steel. Under such conditions, continued spalling is likely and the repair of local spall areas may even promote the deterioration of the concrete because of the presence of dissimilar materials.

An area of steel corrosion and chloride contaminated concrete may be considerably larger than the area of spalled concrete, and the full area of contamination rather than the spall itself shall be broken and replastered.

12.7.1.3 Fire Damage:

Ferrocement may be more susceptible to fire damage than conventional concrete because of the thin cover.

If the fire were intense enough to release the amount of chemically bound water in the cement, destroy the bond between the cement and the aggregate, or oxidize the reinforcement, the surface would be charred and spalled so that the damage could be easily identified. Full scale removal and repair shall then be required.

12.7.1.4 Cracks and Local Fractures:

Hairline cracks and crazing due to temperature changes or drying shrinkage in the cover coat do not require repair. Continuous wet curing will cause autogenous healing, and a flexible coating will conceal the crack from view. If cracks are caused by continuing overloads or are due to structural settlement and the cause cannot be removed, replacement or a structural overlay shall be required. Cracks due to occasional impact or overload may be repaired. Local fractures are cracks in which displacement of the section has occurred as a result of impact.

12.7.2 Evaluation of Damage

12.7.2.1 Evaluation of damage shall take into consideration its extent, cause, and likelihood of the cause still being active. The method of repair shall be dictated by the type of damage, the availability of special equipment and repair materials, and the level of skill of the workers employed. Economic factors may influence the decision as to whether the repair shall be extensive and permanent, or limited in scope in response to an immediate problem.

12.7.2.2 Repair materials shall bond to the original structure, resist pollutants in the surrounding soil, water or air, and respond the same way to changes in temperature, moisture, and loads. Removal of deteriorated or chloride contaminated mortar trapped within the reinforcing mesh requires a large amount of hand labour, so it may be economical (and better for long term durability) to reconstruct or

replace an entire area using the original structure as a form that can be left in place or removed after the overlaid structure has cured. Complete reconstruction shall be undertaken when chloride contamination, mesh corrosion, and deterioration of the mortar are extensive.

- 12.7.2.3 Testing for damage in ferrocement may be done by tapping with a hammer to break into any voids under the surface, or by drawing a metal bar over the surface and listening for sounds indicating voids or the presence of deteriorated concrete. A high quality ferrocement should produce a bell like sound and resist moderately severe hammer blows without damage.

12.7.3 Surface Preparation for Repair of Damage

12.7.3.1 General :

The primary objective is to remove any deteriorated mortar or mortar contaminated with corrosive agents and to provide a surface to which the repair materials can be bonded properly. The rougher the surface, the greater is the area available for bonding.

12.7.3.2 Removal of Deteriorated Concrete:

As a first step in any repair all disintegrated, unsound, and contaminated mortar shall be removed. Saws and chipping hammers used for conventional concrete shall not be used for ferrocement unless large sections are to be completely removed.

Small areas shall be prepared by hand hammering just hard enough to pulverize deteriorated or cracked mortar, but not to the point of damaging the reinforcing mesh.

A pneumatic needle gun may be used for cleaning out broken ferrocement, opening out cracks, and roughening the surface.

Particles of sound mortar embedded in the mesh need not be removed provided they are small enough not to interfere with the penetration of new mortar and they will not project from the finished surface.

12.7.3.3 Reinforcement:

Any loose, scaly corrosion revealed on cleaning out the mortar shall be removed by sandblasting, water jet, air blasting, or vacuum methods.

An alternative method for removing rust is to brush naval jelly or spray dilute phosphoric acid over the repair area and flush thoroughly.

Where the mesh cage has been displaced but is still intact, it may be pushed or jacked back in place and supported securely to withstand the pressure of applying the repair material. Where the reinforcement has been torn, the old mesh shall be laced back to close the opening.

When rods supporting the mesh cage are torn they shall be spliced by a 15 diameter overlap of the partner rod or anchored by hooks.

12.7.3.4 Cleaning:

Loose particles and dust residue from hammering or sandblasting shall be air jetted or vacuum cleaned if epoxy or methymethacrylate (MMA) is the repair material. Water jetting may be used if the repair is to be made with hydraulic cement or latex modified mortar.

If an air jet is used, the compressor shall be equipped with an oil trap to prevent contamination of the surface. Surface oil or dirt shall be removed by trisodium phosphate or other strong detergents.

12.7.3.5 Cracks:

Cracks may be cleaned by hammering out the mortar on each side of the crack and replastered with latex mortar.

If opening the crack is not feasible, epoxy or MMA injection systems shall be attempted in accordance with the product directions. The crack shall be cleaned first with oil free compressed air, and small (about 2 to 3 mm) drill holes shall be made at the highest and lowest points in the crack. The surface between the holes shall be sealed with strong coatings or a pressure pad. Catalyzed epoxy or MMA shall be injected at the lower hole until it comes out at the upper hole.

Where latex cement grout is to be used, the interior of the crack shall be thoroughly saturated with water and allowed to drain.

12.7.4 Repair Materials

12.7.4.1 Portland Cement and Sand:

Portland cement used for repair shall conform to the requirements of Sec 12.3.1.

Sand which matches that used in the original construction may be used unless the need for the repair arose because of reactive or contaminated sand. Neat Portland or blended cement paste shall be used to fill small cracks, and a mortar with fine sand shall be used to fill larger cracks or voids. Both shall be used in combination with latex for thin patches and overlays. Larger cracks shall be coated with neat cement slurry, and then dry packed with a very low water cement ratio mortar.

The addition of latex to Portland cement mortar markedly improves bond to the substrate and the tensile strength of the patch. Of the synthetic latexes, polyvinyl acetate and polyvinylidene are unsuitable for wet environments. Acrylics may be used as admixtures to improve bonding and as curing compounds. Acrylic latex in concentrated form shall be diluted to 10 to 20 per cent solids and then used as the mixing water for the mortar. Latex mortars may be applied to a damp surface, but the patch shall be allowed to dry thoroughly before being immersed in water.

12.7.4.2 Polymer Mortars:

Nonlatex polymer mortars shall require the use of surface dried and, preferably, oven dried sand. The monomers have very low viscosity and so shall be mixed with thickening agents to be placed in any area that cannot be sealed tightly. Epoxy resins that are moisture tolerant may be used on damp surfaces.

Care shall be exercised in applying polymers or the promoters and hardeners used with them which are toxic.

12.7.4.3 Admixture:

Accelerators may be employed where cement alone is the repair material. Since chloride compounds may promote corrosion, nonchloride accelerators shall be preferred for all ferrocement. Emergency repairs of small areas below the waterline with hot plug, which is neat cement moistened to a putty consistency with a concentrated solution of calcium chloride may be permitted. The hot plug may be carried in the hand or in a plastic bag to the site of the leak, pressed into the hole, and held a few minutes until set. Permanent repair shall be accomplished as soon as possible using materials without chlorides.

12.7.5 Repair Procedure

12.7.5.1 Mixing :

Small quantities of materials required for ferrocement repairs may be hand mixed on flat surface or in a tray using premixed dry ingredients. For large quantities, a plaster or pan mixer rather than a rotating drum type mixer shall be used. For machine mixing water shall be put in first; then the cement, to form slurry; then the pozzolan, if used; and finally, enough sand to bring the mortar to the desired degree of workability.

The consistency of the mortar shall be selected according to the nature of the repair. A slurry of cream consistency shall be used first to paint the moistened edges of the repair area, fill cracks or small voids, and thoroughly coat all the interior mesh and rods. After this, more sand shall be added until the mortar is stiff enough to hold its shape when brought out flush with the finished surface.

To avoid excessive amounts of entrained air, mortars containing acrylics or epoxies shall not be mixed longer than two minutes. They shall be applied within thirty minutes of mixing.

12.7.5.2 Full Depth Repair:

When both faces are accessible, a fluid mortar shall be pushed through the mesh cage from one side until an excess appears on the opposite face. This excess shall then be pushed back and finished flush. A vibrating float or trowel may be used to place and finish a very stiff mortar. Pencil type vibrators shall not be used.

12.7.5.3 Partial Depth Patches:

The area to be patched shall first be saturated with water, then air blown or blotted free of standing water until only surface-moist. Cement slurry of not more than 0.4 water cement ratio and of paint like consistency shall be brushed over the whole area and into any openings in the mesh. This shall be immediately followed by a heavily sanded mortar of the same water cement ratio, which shall be vibrated or tamped into the patch and finished flush.

12.7.5.4 **Overlays:**

The substrate shall be prepared in the manner prescribed in Sec 12.7.5.3 for patches. The old surface shall be thoroughly cleaned or scarified by mechanical means and the repair materials shall match the thermal characteristics of the substrate.

Chemical etching shall be followed by mechanical abrasion, unless the surface is flushed with high pressure water jet equipment.

For thin overlays, velocity placement such as spritzing or casting by hand, and shotcreting, shall be used.

12.7.5.5 **Shotcrete:**

Shotcrete may be used in ferrocement repair when a large area is involved. Small, low cost portable plaster pumps operating on the Moyno progressive cavity principle with a rotor inside a stator tube shall be adequate for both original ferrocement construction and repair.

Shotcrete or plastering equipment may be used for large overlays incorporating additional layers of reinforcing mesh by laminating techniques. Existing surfaces shall be scarified or sandblasted, then saturated with water and allowed to damp dry just before the shotcrete or mortar spray is applied. An initial application of cement slurry is not needed with shotcrete but a latex or wet to dry epoxy bonding compound may be used to advantage with repairs made with plastering equipment.

12.7.5.6 **Curing :**

All Portland cement patches and overlays shall be thoroughly cured unless latex compounds are used to seal the surface and furnish water for hydration. Curing shall be instituted immediately for thin patches and overlays. Several layers of paper or cloth soaked in water and covered with a plastic sheet that is well secured at the edges may be used on patches. A full plastic film covering overlays may be used but it may produce discoloration where it touches the surface.

12.8 **TESTING**

12.8.1 **Test Requirement**

Tests and observations that are commonly made during the design, construction, and subsequent service life of concrete structures shall also be applicable to ferrocement structures. The test programme shall include

- a) tests on physical, chemical and mechanical properties of the ferrocement ingredients, such as water purity, sieve analysis, mesh strength etc.,
- b) control tests for fresh mortar mix, such as slump, air content etc.,
- c) tests on the mechanical properties of the hardened ferrocement, such as bending, cracking and fatigue strengths, permeability etc., and
- d) in-service condition tests, such as potential for corrosion, cracking, durability etc.

For predicting the mechanical properties of ferrocement, the tests specified in Sec 12.8.2.1 through 12.8.2.4 shall be conducted.

12.8.2 **Test Methods**

12.8.2.1 **Compressive Strength and Static Modulus of Elasticity of Mortar :**

The compressive strength and static modulus of elasticity of the mortar used for the fabrication of ferrocement shall be determined from 75 mm x 150 mm cylinders tested in accordance with ASTM C39-86 and C469-83, respectively.

12.8.2.2 **Flexural Strength of Ferrocement:**

Ferrocement specimens shall be tested as a simply supported beam with third point loading. The span to depth ratio of the beam specimen shall not be less than 20 and its width shall not be less than six times the mesh opening or wire spacing measured at right angles to the span direction.

12.8.2.3 Tensile Properties of the Mesh Reinforcement:

Square or rectangular meshes may be tested directly in tension; hexagonal meshes and expanded metal meshes shall be tested only while encapsulated in mortar. In the latter case the tensile test shall be performed on the ferrocement material as described in Sec 12.8.2.4 below.

For square and rectangular meshes, the yield strength, elastic modulus, and ultimate tensile strength shall be obtained from direct tensile tests on samples of wires or flat coupons cut from the mesh. The test shall be in accordance with the following guidelines (see also Fig 12.8.1).

1. The test specimen shall be prepared by embedding both ends of a rectangular coupon of mesh in mortar over a length at least equal to the width of the sample. The mortar embedded ends shall serve as pads for gripping. The free (not embedded) portion of the mesh shall represent the test sample.
2. The width of the test sample shall be not less than six times the mesh opening or wire spacing measured at right angles to the loading direction.
3. The length of the test sample shall be not less than three times its width or 150 mm, whichever is larger.
4. Measurements of elongations (from which strains are to be computed) shall be recorded over half the length of the mesh sample.
5. Yield strain of mesh reinforcement shall be taken as the strain at the intersection of the best straight line fit of the initial portion of the stress strain curve and the best straight line fit of the yielded portion of the stress strain curve, as shown in Fig 12.8.1. The yield stress shall be taken as the stress point on the original stress strain curve at the yield strain found above. The procedure is demonstrated in Fig 12.8.1.

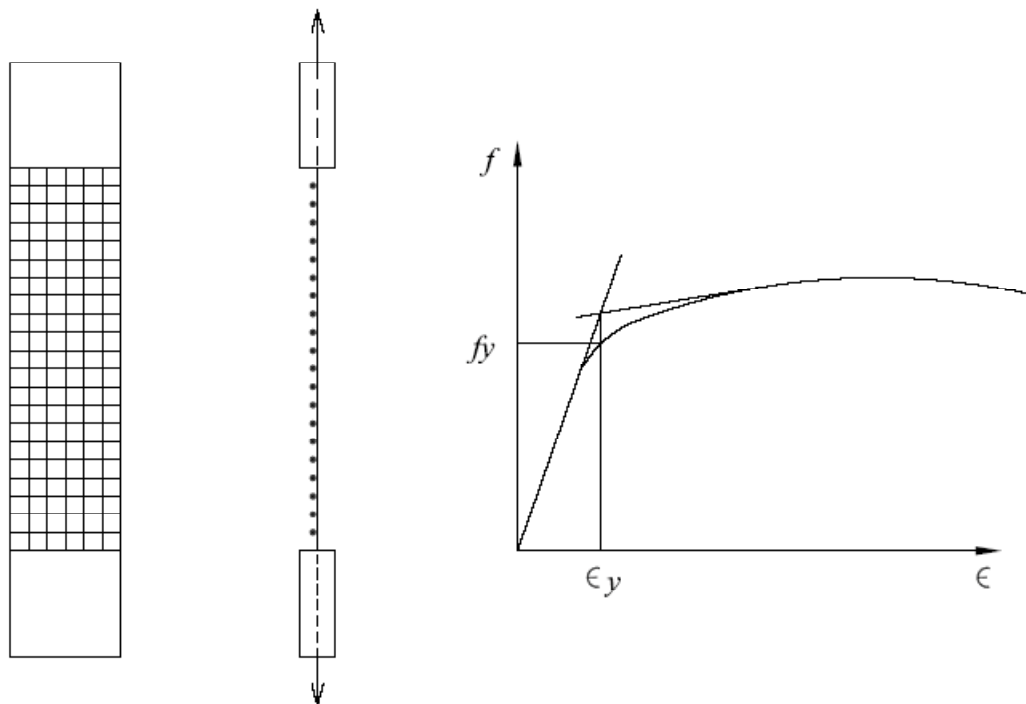


Fig. 12.8.1 Schematic Description of Mesh Tensile Test Sample and Corresponding Stress-Strain Curve

12.8.2.4 Tensile Test of Ferrocement:

Direct tensile tests of ferrocement elements shall be made using rectangular specimens satisfying the same minimum size requirement as those set in Sec 12.8.2.3 for the mesh reinforcement. The test specimens shall be additionally reinforced at their ends for gripping. The middle half of the nongripped (free) portion of the test specimen shall be instrumented to record elongations. A plot of the load elongation curve up to failure shall be

used to estimate the effective modulus of the mesh system as well as its yield strength, ultimate strength, and efficiency factor. The yield strain and corresponding stress shall be determined in accordance with the procedure described in Sec 12.8.2.3.

Related Appendices

Appendix 6.12.A Calculation of Volume Fraction of Reinforcement

Appendix 6.12.B Common Types and Sizes of Steel Meshes Used in Ferrocement

Steel-Concrete Composite Structural Members

13.1 General Provisions for Steel-Concrete Composite Structural Members

This section states the scope of the specification, summarizes referenced specifications, codes and standard documents and provide requirements for materials for steel-concrete composite members. General provisions for composite sections and shear connectors are also included.

13.1.1 Scope

The guidelines included in chapter 13 of part 6 of this code presents the design guidelines for steel concrete composite members frequently used in medium to high rise buildings. This chapter mainly addresses composite columns composed of rolled or built-up structural steel shapes or HSS, and structural concrete acting together, and steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with shear connectors and concrete-encased beams, constructed with or without temporary shores, are included. Seismic provisions for steel-concrete composite members are also provided.

13.1.2 Referenced Specifications, Codes and Standards

The documents referenced in these provisions shall include those listed in Part 6 Chapter 10 Section 2 with the following additions and modifications:

American Society of Civil Engineers

ASCE 3-91 Standard for the Structural Design of Composite Slabs

American Welding Society

AWS D1.1-04 Structural Welding Code- Steel

AWS D1.4-98 Structural Welding Code-Reinforcing Steel

Canadian Standards Association

CSA S16-01 Design of Steel Structures

13.1.3 Material Limitations

Concrete and steel reinforcing bars in *composite* systems shall be subject to the following limitations.

- a) For the determination of the *available strength*, concrete shall have a compressive strength f'_c of not less than 21 MPa nor more than 70 MPa for normal weight concrete and not less than 21 MPa nor more than 42 MPa for lightweight concrete.
- b) The *specified minimum yield stress* of structural steel and reinforcing bars used in calculating the strength of a *composite column* shall not exceed 525 MPa.

Higher material strengths are permitted when their use is justified by testing or analysis.

13.1.4 General Provisions

In determining *load effects* in members and *connections* of a structure that includes *composite* members, consideration shall be given to the effective sections at the time each increment of *load* is applied. The design, detailing and material properties related to the concrete and reinforcing steel portions of composite construction shall comply with the reinforced concrete and reinforcing bar design specifications stipulated by the provisions in Part 6 Chapter 5.

13.1.4.1 Resistance Prior to Composite Action

The factored resistance of the steel member prior to the attainment of composite action shall be determined in accordance with Chapter 10 of Part 6.

13.1.4.2 Nominal Strength of Composite Sections

Two methods are provided for determining the nominal strength of composite sections: the plastic stress distribution method and the strain-compatibility method.

The tensile strength of the concrete shall be neglected in the determination of the nominal strength of composite members.

13.1.4.2.1 Plastic Stress Distribution Method

For the plastic stress distribution method, the nominal strength shall be computed assuming that steel components have reached a *stress* of F_y in either tension or compression and concrete components in compression have reached a *stress* of $0.85 f'_c$. For round HSS filled with concrete, a *stress* of $0.95 f'_c$ is permitted to be used for concrete components in uniform compression to account for the effects of concrete confinement.

13.1.4.2.2 Strain-Compatibility Method

For the strain compatibility method, a linear distribution of strains across the section shall be assumed, with the maximum concrete compressive strain equal to 0.003 mm/mm (in/in). The stress-strain relationships for steel and concrete shall be obtained from tests or from published results for similar materials.

13.1.4.2.3 Shear Connectors

Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot-rolled steel channels. Shear stud design values shall be taken as per Sections 13.2.1.7 and 13.3.2.4. Stud connectors shall conform to the requirements of Section 13.3.2.4(3) Channel connectors shall conform to the requirements of Section 13.3.2.4(4).

13.2 Design of Composite Axial Members

This section states the design guidelines for two types of composite axial members. These include— encased composite columns and concrete filled hollow structural sections.

13.2.1 Encased Composite Columns

13.2.1.1 Scope

This section applies to doubly symmetric steel columns encased in concrete, provided that

- (1) the steel shape is a compact or non-compact section;
- (2) the cross-sectional area of the steel core comprises at least 1 percent of the total composite cross section.
- (3) concrete encasement of the steel core is reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum *transverse reinforcement* shall be at least 6 mm^2 per mm of tie spacing.
- (4) The minimum reinforcement ratio for continuous longitudinal reinforcing,

ρ_{sr} , shall be 0.004, where ρ_{sr} is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \quad 13.2.1$$

Where

A_{sr} = area of continuous reinforcing bars, mm^2

A_g = gross area of composite member, mm^2

13.2.1.2 Compressive Strength

The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c , for axially loaded *encased composite columns* shall be determined for the limit state of *flexural buckling* based on column slenderness as follows:

$$\phi_c = 0.75 \text{ (LRFD)} \quad \Omega_c = 2.00 \text{ (ASD)}$$

$$(a) \text{ When } P_e \geq 0.44P_o$$

$$P_n = P_o \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] \quad 13.2.2$$

$$(b) \text{ When } P_e < 0.44P_o$$

$$P_n = 0.877P_e \quad 13.2.3$$

Where

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c \quad 13.2.4$$

$$P_e = \frac{\pi^2 (EI_{eff})}{(KL)^2} \quad 13.2.5$$

and where

A_s = area of the steel section, mm²

A_c = area of concrete, mm²

A_{sr} = area of continuous reinforcing bars, mm²

E_c = modulus of elasticity of concrete = $0.043 w_c^{1.5} \sqrt{f'_c}$ MPa

E_s = modulus of elasticity of steel = 210 MPa

f'_c = specified compressive strength of concrete, MPa

F_y = specified minimum yield stress of steel section, MPa

F_{yr} = specified minimum yield stress of reinforcing bars, MPa

I_c = moment of inertia of the concrete section, mm⁴

I_s = moment of inertia of steel shape, mm⁴

I_{sr} = moment of inertia of reinforcing bars, mm⁴

K = the effective length factor determined in accordance with Chapter 10 Part 6

L = laterally unbraced length of the member, mm

w_c = weight of concrete per unit volume $1500 \leq w_c \leq 2500$ kg/m³

Where

EI_{eff} = effective stiffness of composite section, N-mm²

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \quad 13.2.6$$

Where

$$C_1 = 0.1 + 2 \left(\frac{A_s}{A_c + A_s} \right) \leq 0.3 \quad 13.2.7$$

13.2.1.3 Tensile Strength

The *design tensile strength*, $\phi_t P_n$, and *allowable tensile strength*, P_n / Ω_t , for encased composite columns shall be determined for the limit state of *yielding* as

$$P_n = A_s F_y + A_{sr} F_{yr} \quad 13.2.8$$

$$\phi_t = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)}$$

13.2.1.4 Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Section 10.7 plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone.

13.2.1.5 Load Transfer

Loads applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

- When the external *force* is applied directly to the steel section, *shear connectors* shall be provided to transfer the required shear force, V' , as follows:

$$V' = V \left(1 - \frac{A_s F_y}{P_o} \right) \quad 13.2.9$$

Where

V = required shear force introduced to *column*, N

A_s = area of steel cross section, mm²

P_o = nominal axial compressive strength without consideration of length effects, N

When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the required shear force, V' , as follows:

$$V' = V \left(\frac{A_s F_y}{P_o} \right) \quad 13.2.10$$

- When load is applied to the concrete of an encased composite column by direct bearing the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p / Ω_B , of the concrete shall be:

$$P_p = 1.7 f'_c A_B \quad 13.2.11$$

Where

$$\phi_B = 0.65 \text{ (LRFD)} \quad \Omega_B = 2.31 \text{ (ASD)}$$

where

A_B = loaded area of concrete, mm²

13.2.1.6 Detailing Requirements

13.2.1.6.1 Longitudinal Bars

The concrete encasement shall be reinforced with longitudinal bars and lateral ties extending completely around the structural steel core. The clear cover shall not be less than 40 mm.

The longitudinal bars shall

- Be continuous at framed levels when considered to carry load;
- Have an area not less than 0.01 times the total gross cross-sectional area;
- Be located at each corner; and
- Spaced on all sides not further apart than 525t/fy times one-half the least dimension of the composite section.

13.2.1.6.2 Lateral ties

The lateral ties shall

- a) Be 15M bars, except that 10M bars may be used when no side dimension of the composite section exceeds 500 mm; and
- b) Have a vertical spacing not exceeding the least of the following:
 - i) Two-thirds of the least side dimension of the cross-section;
 - ii) 16 longitudinal bar diameters; or
 - iii) 500 mm.

Where required, shear connectors transferring the required shear force shall be distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region. The maximum connector spacing shall be 405 mm.

13.2.1.6.3 Shear Connectors

Shear connectors shall be provided to transfer the required shear force. Section specified in 13.2.2.5. The shear connectors shall be distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region. The maximum connector spacing shall be 405 mm. Connectors to transfer axial load shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

13.2.1.6.4 Columns with Multiple Built-up Shapes

If the composite cross section is built up from two or more encased steel shapes, the shapes shall be interconnected with *lacing*, *tie plates*, *batten plates* or similar components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

13.2.1.7 Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete is:

$$Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \leq A_{sc}F_u \quad 13.2.12$$

where

A_{sc} = cross-sectional area of stud shear connector, mm²

F_u = specified minimum tensile strength of a stud shear connector, MPa

13.2.2 Concrete Filled Hollow Structural Section

13.2.2.1 Scope

Section 13.2.2 applies to composite members consisting of steel hollow structural sections completely filled with concrete, provided that

- a) The cross-sectional area of the steel HSS shall comprise at least 1 percent of the total composite cross-section.
- b) The width-to thickness ratio of the walls of rectangular hollow structural sections does not exceed $1350/\sqrt{F_y}$
- c) The outside diameter-to-thickness ration of circular hollow structural sections does not exceed $28000/F_y$
- d) The concrete strength is between 20 and 80 MPa for axially loaded columns and between 20 and 40 MPa for columns subjected to axial compression and bending.

13.2.2.2 Compressive Strength

The *design compressive strength*, $\phi_c P_n$ and *allowable compressive strength*, P_n/Ω_c , for axially loaded filled composite columns shall be determined for the *limit state of flexural buckling* based on Section 13.2.1.2 with the following modifications:

$$P_o = A_s F_y + A_{sr} F_{yr} + C_2 A_c f'_c \quad 13.2.13$$

$C_2 = 0.85$ for rectangular sections and 0.95 for circular sections

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \quad 13.2.14$$

$$C_3 = 0.6 + 2 \left(\frac{A_s}{A_c + A_s} \right) \leq 0.9 \quad 13.2.15$$

13.2.2.3 Tensile Strength

The *design tensile strength*, $\phi_t P_n$, and allowable tensile strength, P_n / Ω_t , for filled composite columns shall be determined for the limit state of *yielding* as:

$$P_n = A_s F_y + A_{sr} F_{yr} \quad 13.2.16$$

$$\phi_t = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)}$$

13.2.2.4 Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Chapter 10 or the shear strength of the reinforced concrete portion alone. The shear strength of reinforced concrete portion may be determined according to Chapter 6 of Part 6.

13.2.2.5 Load Transfer

Loads applied to filled composite columns shall be transferred between the steel and concrete. When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core is required from *direct bond interaction*, *shear connection* or *direct bearing*. The force transfer *mechanism* providing the largest *nominal strength* may be used. These force transfer mechanisms shall not be superimposed.

When load is applied to the concrete of an encased or filled composite column by *direct bearing* the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p / Ω_B , of the concrete shall be:

$$P_p = 1.7 f'_c A_E \quad 13.2.17$$

$$\phi_B = 0.65 \text{ (LRFD)} \quad \Omega_B = 2.31 \text{ (ASD)}$$

where

A_E is the loaded area, mm^2

13.2.2.6 Detailing Requirements

Where required, shear connectors transferring the required shear force shall be distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region. The maximum connector spacing shall be 405 mm.

13.3 Design of Composite Flexural Members

This section applies to composite beams consisting of steel sections interconnected with either a reinforced concrete slab or a steel deck with a concrete cover slab. The steel *beams* and the reinforced concrete slab are so interconnected that the beams and the slab act together to resist bending. Simple and continuous *composite beams* with *shear connectors* and *concrete-encased beams*, constructed with or without temporary shores, are included. Design philosophy for composite columns subjected to bending moments is also stated.

13.3.1 General

13.3.1.1 Deflections

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 the 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 and interfacial slip, creep, and shrinkage may be assessed as follows:

- a) For increased flexibility resulting from partial shear connection and interfacial slip, the deflections shall be calculated using an effective moment of inertia given by

$$I_e = I_s + 0.85p^{0.25}(I_t - I_s) \quad 13.3.1$$

Where

I_s = moment of inertia of a steel beam, or of a steel joist or truss adjusted to include the effect of shear deformations, which may be taken into account by decreasing the moment of inertia based on the cross-sectional areas of the top and bottom chords by 15% or by a more detailed analysis

P = fraction of full shear connection

=1.00 for full shear connection

I_t = transformed moment of inertia of composite beam based on the modular ration $n=E/E_c$

- b) For creep, elastic deflections caused by dead loads and long-term live loads, as calculated in Item (a), need to be increased by 15% and
- c) For shrinkage of concrete, using a selected free shrinkage strain, strain compatibility between the steel and concrete, and an age-adjusted effective modulus of elasticity of concrete as it shrinks and creeps, the deflection of a simply supported composite beam, joist, or truss shall be calculated as follows:

$$\Delta_s = \frac{L^2}{8} \psi = \frac{L^2}{8} c \frac{\epsilon_f A_c Y}{n_s I_{es}} \quad 13.3.2$$

Where

L = span of the beam, joist, or truss

ψ = curvature along length of the beam, joist, or truss due to shrinkage of concrete

c = empirical coefficient used to match theory with test results (accounting for cracking of concrete in tension, the non-linear stress-strain relationship of concrete, and other factors)

= free shrinkage strain of concrete

A_c = effective area of concrete slab

Y = distance from centroid of effective of effective area of concrete slab to centroidal axis of the composite beam, joist, or truss

n_s = modular ration, E/E'_c

where

$$E'_c = E_c / (1 + \chi \phi) \quad 13.3.3$$

= age-adjusted effective modulus of elasticity of concrete

Where

χ = aging coefficient of concrete

ϕ = creep coefficient of concrete

$$I_{es} = I_s + 0.85p^{0.25}(I_{ts} - I_s) \quad 13.3.4$$

= effective moment of inertia of composite beam, truss, or joist based on the modular ratio n_s

Where

I_{ts} = transformed moment of inertia based on the modular ratio n_s

13.3.1.2 Design Effective Width of Concrete

The *effective width* of the concrete slab is the sum of the effective widths for each side of the beam centerline, each of which shall not exceed:

- one-eighth of the beam span, center-to-center of supports;
- one-half the distance to the centerline of the adjacent beam; or
- the distance to the edge of the slab.

13.3.1.3 Shear Strength

The available shear strength of *composite beams* with *shear connectors* shall be determined based upon the properties of the steel section alone in accordance with Part 6 Chapter 10 Section 10.7. The available shear strength of concrete-encased and filled composite members shall be determined based upon the properties of the steel section alone in accordance with Part 6 Chapter 10 Section 10.7 or based upon the properties of the concrete and longitudinal steel reinforcement.

13.3.1.4 Strength during Construction

When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all *loads* applied prior to the concrete attaining 75 percent of its specified strength f'_c . The available flexural strength of the steel section shall be determined according to Part 6 Chapter 10 Section 10.6.

13.3.2 Strength of Composite Beams with Shear Connectors

13.3.2.1 Positive Flexural Strength

The design positive flexural strength, $\phi_b M_n$, and the allowable positive flexural strength, M_n / Ω_b , shall be determined for the limit state of yielding as follows:

$\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)

- For $h/t_w \leq 3.76 \sqrt{E/F_y}$

M_n shall be determined from the plastic *stress* distribution on the *composite* section for the limit state of *yielding (plastic moment)*.

- For $h/t_w > 3.76 \sqrt{E/F_y}$

M_n shall be determined from the superposition of elastic stresses, considering the effects of shoring, for the limit state of *yielding (yield moment)*.

13.3.2.2 Negative Flexural Strength

The *design negative flexural strength*, $\phi_b M_n$, and the *allowable negative flexural strength*, M_n / Ω_b , shall be determined for the steel section alone, in accordance with the requirements of Part 6 Chapter 10 Section 10.6.

Alternatively, the available negative flexural strength shall be determined from the plastic stress distribution on the composite section, for the limit state of yielding (plastic moment), with

$\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)

provided that:

- The steel *beam* is compact and is adequately braced according to Section 10.6.
- Shear connectors* connect the slab to the steel beam in the negative moment region.
- The slab reinforcement parallel to the steel beam, within the *effective width* of the slab, is *properly developed*.

13.3.2.3 Strength of Composite Beams with Formed Steel Deck

(1) General

The *available flexural strength* of composite construction consisting of concrete slabs on *formed steel deck* connected to steel beams shall be determined by the applicable portions of Section 13.3.2.1 and 13.3.2.2, with the following requirements:

- This section is applicable to decks with *nominal rib height* not greater than 75 mm. The average width of concrete rib or haunch, w_r , shall be not less than 50 mm, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.
- The concrete slab shall be connected to the steel beam with welded stud shear connectors 19 mm or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel cross section. Stud shear connectors, after installation, shall extend not less than 38 mm above the top of the steel deck and there shall be at least 13 mm of concrete cover above the top of the installed studs.
- The slab thickness above the steel deck shall be not less than 50 mm.
- Steel deck shall be anchored to all supporting members at a spacing not to exceed 460 mm. Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

(2) Deck Ribs Oriented Perpendicular to Steel Beam

Concrete below the top of the steel deck shall be neglected in determining composite section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

(3) Deck Ribs Oriented Parallel to Steel Beam

Concrete below the top of the steel deck may be included in determining composite section properties and shall be included in calculating A_c .

Formed steel deck ribs over supporting beams may be split longitudinally and separated to form a *concrete haunch*.

When the nominal depth of steel deck is 38 mm or greater, the average width, w_r , of the supported haunch or rib shall be not less than 50 mm for the first stud in the transverse row plus four stud diameters for each additional stud.

13.3.2.4 Shear Connectors

(1) Load Transfer for Positive Moment

The entire *horizontal shear* at the interface between the steel *beam* and the concrete slab shall be assumed to be transferred by shear connectors, except for *concrete-encased beams* as defined in Section 10.9.3.3. For *composite* action with concrete subject to flexural compression, the total horizontal shear force, V' , between the point of maximum positive moment and the point of zero moment shall be taken as the lowest value according to the *limit states* of *concrete crushing*, *tensile yielding* of the steel section, or strength of the shear connectors:

Concrete crushing

$$V' = 0.85 f'_c A_c \quad 13.3.5a$$

Tensile yielding of the steel section

$$V' = F_y A_s \quad 13.3.5b$$

Strength of shear connectors

$$V' = \sum Q_n \quad 13.3.5c$$

where

A_c = area of concrete slab within *effective width*, mm²

A_s = area of steel cross section, mm^2

$\sum Q_n$ = sum of *nominal strengths* of shear connectors between the point of maximum positive moment and the point of zero moment, N

(2) Load Transfer for Negative Moment

In continuous *composite beams* where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel *beam*, the total *horizontal shear force* between the point of maximum negative moment and the point of zero moment shall be taken as the lower value according to the limit states of *yielding* of the steel reinforcement in the slab, or strength of the shear connectors:

- a) Tensile yielding of the slab reinforcement

$$V' = A_r F_{yr} \quad 13.3.6a$$

where

A_r = area of adequately developed longitudinal reinforcing steel within the *effective width* of the concrete slab, mm^2

F_{yr} = *specified minimum yield stress* of the reinforcing steel, MPa

- b) Strength of shear connectors

$$V' = \sum Q_n \quad 13.3.6b$$

(3) Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete or in a *composite slab* is

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u \quad 13.3.7$$

where

A_{sc} = cross-sectional area of stud *shear connector*, mm^2

E_c = modulus of elasticity of concrete = $0.043 w_c^{1.5} \sqrt{f'_c}$, MPa

F_u = *specified minimum tensile strength* of a stud shear connector

R_g = 1.0; (a) for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for any number of studs welded in a row directly to the steel shape; (c) for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the *average rib width* to rib depth ≥ 1.5

= 0.85; (a) for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for one stud welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth < 1.5

= 0.7 for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape

R_p = 1.0 for studs welded directly to the steel shape (in other words, not through steel deck or sheet) and having a haunch detail with not more than 50 percent of the top flange covered by deck or sheet steel closures

= 0.75; (a) for studs welded in a composite slab with the deck oriented perpendicular to the beam and $e_{mid-ht} \geq 50$ mm; (b) for studs welded through steel deck, or steel sheet used as *girder filler* material and embedded in a *composite slab* with the deck oriented parallel to the *beam*

= 0.6 for studs welded in a composite slab with deck oriented perpendicular to the beam and $e_{mid-ht} < 50$ mm

e_{mid-ht} = distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the *load bearing* direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), mm

w_c = weight of concrete per unit volume ($1500 \leq w_c \leq 2500 \text{ kg/m}^3$)

(4) Strength of Channel Shear Connectors

The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_c E_c} \quad 13.3.8$$

where

t_f = flange thickness of channel shear connector, mm

t_w = web thickness of channel shear connector, mm

L_c = length of channel shear connector, mm

The strength of the channel shear connector shall be developed by welding the channel to the beam flange for a force equal to Q_n , considering eccentricity on the connector.

(5) Required Number of Shear Connectors

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the *horizontal shear force* as determined in Sections 10.9.3.2d(1) and 10.9.3.2d(2) divided by the nominal strength of one shear connector as determined from Section 10.9.3.2d(3) or Section 10.9.3.2d(4).

(6) Shear Connector Placement and Spacing

Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless otherwise specified. However, the number of shear connectors placed between any *concentrated load* and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the *concentrated load* point.

Shear connectors shall have at least 25 mm of lateral concrete cover, except for connectors installed in the ribs of *formed steel decks*. The diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded, unless located over the web. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting *composite beam* and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness nor 900 mm.

13.3.3 Slab Reinforcement

13.3.3.1 General

Slabs shall be adequately reinforced to support all loads and to control both cracking transverse to the composite beam span and longitudinal cracking over the steel section. Reinforcement shall not be less than that required by the specified fire-resistance design of the assembly.

13.3.3.2 Parallel reinforcement

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 that is in compression. The reinforcement of slabs that are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention. Reinforcement at the ends of beams supporting ribbed slabs perpendicular to the beam shall be not less than two 15M bars or equivalent.

13.3.3.3 Transverse reinforcement-concrete slab on metal deck

Unless it is known from experience that longitudinal cracking caused by composite action directly over the steel section is unlikely, additional transverse reinforcement or other effective means shall be provided. 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 0.002 times the concrete area being reinforced and shall be uniformly distributed.

13.3.3.4 Transverse reinforcement- Ribbed slabs

- a) Where the ribs are parallel to the beam span, the area of transverse reinforcement shall be not less than 0.002 times the concrete cover slab area being reinforced and shall be uniformly distributed.
- b) Where the ribs are perpendicular to the beam span, the area of transverse reinforcement shall be not less than 0.001 times the concrete cover slab area being reinforced and shall be uniformly distributed.

13.3.4 Flexural Strength of Concrete-Encased and Filled Members

The *nominal flexural strength* of concrete-encased and filled members shall be determined using one of the following methods:

- a) The superposition of elastic *stresses* on the composite section, considering the effects of shoring, for the *limit state of yielding (yield moment)*, where

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

- b) The plastic stress distribution on the steel section alone, for the *limit state of yielding (plastic moment)*, where

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

- c) If *shear connectors* are provided and the concrete meets the requirements of Section 10.9.1.2, the nominal flexural strength shall be computed based upon the plastic stress distribution on the composite section or from the strain compatibility method, where

$$\phi_b = 0.85 \text{ (LRFD)} \quad \Omega_b = 1.76 \text{ (ASD)}$$

13.3.5 Combined Axial Force and Flexure

The interaction between axial forces and flexure in composite members shall account for stability as required by Chapter C. The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n / Ω_c and the *design flexural strength*, $\phi_b M_n$ and *allowable flexural strength*, M_n / Ω_b , are determined as follows:

$$\phi_c = 0.75 \text{ (LRFD)} \quad \Omega_c = 2.00 \text{ (ASD)}$$

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

(1) The *nominal strength* of the cross section of a *composite* member subjected to combined axial compression and flexure shall be determined using either the *plastic stress distribution method* or the *strain-compatibility method*.

(2) To account for the influence of length effects on the axial strength of the member, the nominal axial strength of the member shall be determined by Section 10.9 with P_o taken as the nominal axial strength of the cross section determined in Section 10.9.4(13) above.

13.3.6 Special Cases

When *composite* construction does not conform to the requirements of Section 13.2 and Section 13.3, the strength of *shear connectors* and details of construction shall be established by testing.

13.4 Composite Connections

This Section is applicable to connections in buildings that utilize composite or dual steel and concrete systems. Composite connections shall be demonstrated to have Design Strength, ductility and toughness that is comparable to that exhibited by similar structural steel or reinforced concrete connections that meet the requirements in Part 6 Chapter 10 and Chapter 5, respectively. Methods for calculating the connection strength shall meet the requirements in this Section.

13.4.1 General

Connections shall have adequate deformation capacity to resist the critical Required Strengths at the Design Story Drift. Additionally, connections that are required for the lateral stability of the building under seismic forces shall meet the requirements in Section 13.5 based upon the specific system in which the connection is used. When the Required Strength is based upon nominal material strengths and nominal member dimensions, the determination of the required connection strength shall account for any effects that result from the increase in the actual Nominal Strength of the connected member.

13.4.2 Nominal Strength of Connections

The Nominal Strength of connections in composite Structural Systems shall be determined on the basis of rational models that satisfy both equilibrium of internal forces and the strength limitation of component materials and elements based upon potential limit states. Unless the connection strength is determined by analysis and testing, the models used for analysis of connections shall meet the following requirements:

(1) When required, force shall be transferred between structural steel and reinforced concrete through direct bearing of headed shear studs or suitable alternative devices, by other mechanical means, by shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer, or by a combination of these means. Any potential bond strength between structural steel and reinforced concrete shall be ignored for the purpose of the connection force transfer mechanism.

(2) The nominal bearing and shear-friction strengths shall meet the requirements in Part 6 Chapter 6 and 10, except that the strength reduction (resistance) factors shall be as given in Part 6 Chapter 6. Unless a higher strength is substantiated by cyclic testing, the nominal bearing and shear-friction strengths shall be reduced by 25 percent for the composite seismic systems.

(3) The Design Strengths of structural steel components in composite connections, as determined in Section 13.2 and Section 13.3 and the LRFD Specification, shall equal or exceed the Required Strengths. Structural steel elements that are encased in confined reinforced concrete are permitted to be considered to be braced against out of plane buckling. Face Bearing Plates consisting of stiffeners between the flanges of steel beams are required when beams are embedded in reinforced concrete columns or walls.

(4) The nominal shear strength of reinforced-concrete-encased steel Panel Zones in beam-to-column connections shall be calculated as the sum of the Nominal Strengths of the structural steel and confined reinforced concrete shear elements as determined in Part 6 Chapter 10 and Part 6 Chapter 5, respectively. The strength reduction (resistance) factors for reinforced concrete shall be as given in Part 6 Chapter 6.

(5) Reinforcement shall be provided to resist all tensile forces in reinforced concrete components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as appropriate, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with Part 6 Chapter 6. Connections shall meet the following additional requirements:

(a) When the slab transfers horizontal diaphragm forces, the slab reinforcement shall be designed and anchored to carry the in-plane tensile forces at all critical sections in the slab, including connections to collector beams, columns, braces and walls.

(b) For connections between structural steel or Composite Beams and reinforced concrete or Reinforced-Concrete-Encased Composite Columns, transverse hoop reinforcement shall be provided in the connection region to meet the requirements in Chapter 6 of Part 6 except for the following modifications:

(i) Structural steel sections framing into the connections are considered to provide confinement over a width equal to that of face bearing stiffener plates welded to the beams between the flanges.

(ii) Lap splices are permitted for perimeter ties when confinement of the splice is provided by Face Bearing Plates or other means that prevents spalling of the concrete cover.

(c) The longitudinal bar sizes and layout in reinforced concrete and Composite Columns shall be detailed to minimize slippage of the bars through the beam-to-column connection due to high force transfer associated with the change in column moments over the height of the connection.

13.5 Seismic Provisions for Composite Structural Systems

These Provisions are intended for the design and construction of composite structural steel and reinforced concrete members and connections in the Seismic Load Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response.

13.5.1 Scope

Provisions shall be applied in conjunction with the AISC Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings, hereinafter referred to as the LRFD Specification. All members and connections in the Seismic Load Resisting System shall have a Design Strength as required in the LRFD Specification and shall meet the requirements in these Provisions. The applicable requirements in Part 6 Chapter 10 shall be used for the design of structural steel components in composite systems. Reinforced-concrete members subjected to seismic forces shall meet the requirements in Chapter 5 and 10 of Part 6 except as modified in these provisions. When the design is based upon elastic analysis, the stiffness properties of the component members of composite systems shall reflect their condition at the onset of significant yielding of the building.

13.5.2 Seismic Design Categories

The Required Strength and other seismic provisions for Seismic Design Categories, Seismic Use Groups or Seismic Zones and the limitations on height and irregularity shall be as stipulated in the Part 6 Chapter 10.

13.5.3 Loads, Load Combinations, and Nominal Strengths

The loads and load combinations shall be as stipulated by the Applicable Building Code. Where Amplified Seismic Loads are required by these provisions, the horizontal earthquake load E (as defined in Part 6 Chapter 10) shall be multiplied by the over strength factor Ω_o prescribed by the Part 6 Chapter 10.

13.5.4 Materials

13.5.4.1 Structural Steel

Structural steel used in composite Seismic Load Resisting Systems shall meet the requirements in Section 10.20 of Part 6 in addition Section 13.1 of Part 6. The structural steels that are explicitly permitted for use in seismic design have been selected based upon their inelastic properties and weld ability. In general, they meet the following characteristics: (1) a ratio of yield stress to tensile stress not greater than 0.85; (2) a pronounced stress-strain plateau at the yield stress; (3) a large inelastic strain capability (for example, tensile elongation of 20 percent or greater in a 2-in. (50 mm) gage length); and (4) good weldability. Other steels should not be used without evidence that the above criteria are met.

13.5.4.2 Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite Seismic Load Resisting Systems shall meet the requirements in Part 6 Chapter 5, and the following requirements:

(1) The specified minimum compressive strength of concrete in composite members shall equal or exceed 2.5 ksi (17 MPa).

(2) For the purposes of determining the Nominal Strength of composite members, f'_c shall not be taken as greater than 10 ksi (69 MPa) for normal-weight concrete nor 4 ksi (28 MPa) for lightweight concrete.

Concrete and steel reinforcement used in the composite Seismic Load Resisting Systems described shall also meet the requirements in Part 6 Chapter 6.

13.5.5 Composite Members

13.5.5.1 Composite Floor and Roof Slabs

The design of composite floor and roof slabs shall meet the requirements of ASCE 3-91. Composite slab diaphragms shall meet the requirements in this Section.

Details shall be designed to transfer forces between the diaphragm and Boundary Members, Collector Elements, and elements of the horizontal framing system.

The nominal shear strength of composite diaphragms and concrete-filled steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with Part 6 Chapter 5. Alternatively, the composite diaphragm design shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

13.5.5.2 Composite Beams

Composite Beams shall meet the requirements in Section 13.3. Composite Beams that are part of C-SMF shall also meet the following requirements:

- (1) The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed:

$$\frac{Y_{con} + d_b}{1 + \left(\frac{1700 F_y}{E_s} \right)} \quad 13.5.1$$

where

Y_{con} = distance from the top of the steel beam to the top of concrete, mm

d_b = depth of the steel beam, mm

F_y = specified minimum yield strength of the steel beam, MPa

E_s = modulus of elasticity of the steel beam, MPa

- (2) Beam flanges shall meet the requirements in Part 6 Section 10.20.9.4.2, except when fully reinforced-concrete-encased compression elements have a reinforced concrete cover of at least 2 in. (50 mm) and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement shall meet the requirements in Part 6 Chapter 6.

13.5.5.3 Reinforced Concrete Encased Composite Columns

This Section is applicable to columns that: (1) consist of reinforced-concrete encased structural steel sections with a structural steel area that comprises at least 4 percent of the total composite-column cross-section; and (2) meet the additional limitations in Section 13.2.2.1. Such columns shall meet the requirements in Section 13.2.2, except as modified in this Section. Additional requirements, as specified for intermediate and special seismic systems in Sections 13.5.5.3.2 and 13.5.5.3.3, shall apply as required.

Columns that consist of reinforced-concrete-encased structural steel sections with a structural steel area that comprises less than 4 percent of the total composite column cross-section shall meet the requirements for reinforced concrete columns in Part 6 Chapter 5 except as modified for:

- (1) The steel shape shear connectors in Section 13.5.4.3.1 (2).
- (2) The contribution of the reinforced-concrete-encased structural steel section to the strength of the column as provided in Part 6 Chapter 6.
- (3) The seismic requirements for reinforced concrete columns as specified in the description of the composite seismic systems in Sections 13.5.5.3.1 through 13.5.5.3.3.

13.5.5.3.1 Ordinary Seismic System Requirements

The following requirements for Reinforced-Concrete-Encased Composite Columns are applicable to all composite systems:

- (1) The nominal shear strength of the column shall be determined as the nominal shear strength of the structural shape plus the nominal shear strength that is provided by the tie reinforcement in the reinforced-concrete encasement. The nominal shear strength of the structural steel section shall be determined in accordance with Section 10.20 of Chapter 6. The nominal shear strength of the tie reinforcement shall be determined in accordance with Part 6 Chapter 5. In Part 6 Chapter 5, the dimension b_w shall equal the width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear. The nominal shear strength shall be multiplied by ϕ_v equal to 0.75 to determine the design shear strength.

- (2) Composite Columns that are designed to share the applied loads between the structural steel section and reinforced concrete shall have shear connectors that meet the following requirements:

(a) If an external member is framed directly to the structural steel section to transfer a vertical reaction V_u , shear connectors shall be provided to transfer the force $V_u(1 - A_s F_y / P_n)$ between the structural steel section and the reinforced concrete, where A_s is the area of the structural steel section, F_y is the specified minimum yield strength of the structural steel section, and P_n is the nominal compressive strength of the Composite Column.

(b) If an external member is framed directly to the reinforced concrete to transfer a vertical reaction V_u , shear connectors shall be provided to transfer the force $V_u A_s F_y / P_n$ between the structural steel section and the reinforced concrete, where A_s , F_y and P_n are as defined above.

(c) The maximum spacing of shear connectors shall be 16 in. (406 mm) with attachment along the outside flange faces of the embedded shape.

(3) The maximum spacing of transverse ties shall be the least of the following:

(a) one-half the least dimension of the section

(b) 16 longitudinal bar diameters

(c) 48 tie diameters

Transverse ties shall be located vertically within one-half the tie spacing above the top of the footing or lowest beam or slab in any story and shall be spaced as provided herein within one-half the tie spacing below the lowest beam or slab framing into the column.

Transverse bars shall have a diameter that is not less than one-fiftieth of greatest side dimension of the composite member, except that ties shall not be smaller than No. 3 bars and need not be larger than No. 5 bars. Alternatively, welded wire fabric of equivalent area is permitted as transverse reinforcement except when prohibited for intermediate and special systems.

(4) All Load-Carrying Reinforcement shall meet the detailing and splice requirements in Part 6 Chapter 5. Load-Carrying Reinforcement shall be provided at every corner of a rectangular cross-section. The maximum spacing of other load carrying or restraining longitudinal reinforcement shall be one-half of the least side dimension of the composite member.

(5) Splices and end bearing details for reinforced-concrete-encased structural steel sections shall meet the requirements in Chapter 5 of Part 6. If adverse behavioral effects due to the abrupt change in member stiffness and nominal tensile strength occur when reinforced-concrete encasement of a structural steel section is terminated, either at a transition to a pure reinforced concrete column or at the Column Base, they shall be considered in the design.

13.5.5.3.2 Intermediate Seismic System Requirements

Reinforced-Concrete-Encased Composite Columns in intermediate seismic systems shall meet the following requirements in addition to those in Section 13.5.5.3.1:

(1) The maximum spacing of transverse bars at the top and bottom shall be the least of the following:

a) one-half the least dimension of the section

b) 8 longitudinal bar diameters

c) 24 tie bar diameters

d) 12 in. (305 mm)

These spacings shall be maintained over a vertical distance equal to the greatest of the following lengths, measured from each joint face and on both sides of any section where flexural yielding is expected to occur:

a) one-sixth the vertical clear height of the column

b) the maximum cross-sectional dimension

c) 18 in. (457 mm)

(2) Tie spacing over the remaining column length shall not exceed twice the spacing defined above.

(3) Welded wire fabric is not permitted as transverse reinforcement in intermediate seismic systems.

13.5.5.3.3 Special Seismic System Requirements

Reinforced-concrete-encased columns for special seismic systems shall meet the following requirements in addition to those in Sections 13.5.4.3.2 and Sections 13.5.4.4.3:

(1) The required axial strength for Reinforced-Concrete-Encased Composite Columns and splice details shall meet the requirements in Section 13.2.

(2) Longitudinal Load-Carrying Reinforcement shall meet the requirements in Part 6 Chapter 6.

(3) Transverse reinforcement shall be hoop reinforcement as defined in Part 6 Chapter 6 and shall meet the following requirements:

- a) The minimum area of tie reinforcement A_{sh} shall meet the following requirement:

$$A_{sh} = 0.09h_{cc}s \left(1 - \frac{F_y A_s}{P_n} \right) \left(\frac{f'_c}{F_{yh}} \right) \quad 13.5.2$$

where

h_{cc} = cross-sectional dimension of the confined core measured center-to-center of the tie reinforcement, *mm*

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, *mm*

F_y = specified minimum yield strength of the structural steel core, *MPa*

A_s = cross-sectional area of the structural core, *mm*²

P_n = nominal axial compressive strength of the Composite Column calculated in accordance with the LRFD Specification, *N*

f'_c = specified compressive strength of concrete, *MPa*

F_{yh} = specified minimum yield strength of the ties, *MPa*

Equation 13.5.2 need not be satisfied if the Nominal Strength of the reinforced concrete encased structural steel section alone is greater than 1.0D + 0.5L.

- b) The maximum spacing of transverse reinforcement along the length of the column shall be the lesser of 6 longitudinal load-carrying bar diameters and 152 mm (6 in.).
- c) When specified in Sections 13.5.5.3.3 (4), (5) or (6), the maximum spacing of transverse reinforcement shall be the lesser of one-fourth the least member dimension and 102 mm (4 in.). For this reinforcement, cross ties, legs of overlapping hoops, and other confining reinforcement shall be spaced not more than 355 mm (14 in.) on center in the transverse direction.

(4) Reinforced-Concrete-Encased Composite Columns in Braced Frames with axial compression forces that are larger than 0.2 times P_o shall have transverse reinforcement as specified in Section 13.5.5.3.3(3), over the total element length. This requirement need not be satisfied if the Nominal Strength of the reinforced-concrete-encased steel section alone is greater than 1.0D + 0.5L.

(5) Composite Columns supporting reactions from discontinued stiff members, such as walls or Braced Frames, shall have transverse reinforcement as specified in Section 13.5.5.3.3(3)(c) over the full length beneath the level at which the discontinuity occurs if the axial compression force exceeds 0.1 times P_o . Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the reinforced-concrete-encased structural steel section and longitudinal reinforcement. This requirement need not be satisfied if the Nominal Strength of the reinforced-concrete-encased structural steel section alone is greater than 1.0D + 0.5L.

(6) Reinforced-Concrete-Encased Composite Columns that are used in C-SMF shall meet the following requirements:

- a) Transverse reinforcement shall meet the requirements in 13.5.5.3.3(3)(c) at the top and bottom of the column over the region specified in Section 6.4b.
- b) The strong-column/weak-beam design requirements in shall be satisfied. Column Bases shall be detailed to sustain inelastic flexural hinging.
- c) The minimum required shear strength of the column shall meet the requirements in Part 6 Chapter 5.

(7) When the column terminates on a footing or mat foundation, the transverse reinforcement as specified in this section shall extend into the footing or mat at least 305 mm (12 in.). When the column terminates on a wall, the transverse reinforcement shall extend into the wall for at least the length required to develop full yielding in the reinforced-concrete-encased structural steel section and longitudinal reinforcement.

(8) Welded wire fabric is not permitted as transverse reinforcement for special seismic systems.

13.5.5.4 Concrete Filled Composite Columns

This Section is applicable to columns that: (1) consist of concrete-filled steel rectangular or circular hollow structural sections (HSS) with a structural steel area that comprises at least 4 percent of the total composite-column cross-section; and (2) meet the additional limitations in Section 13.2. Such columns shall be designed to meet the requirements in Section 13.2, except as modified in this Section.

The design shear strength of the Composite Column shall be the design shear strength of the structural steel section alone.

In the special seismic systems described in, members and column splices for Concrete-Filled Composite Columns shall also meet the requirements in Part 6 Section 10.20.

Concrete-Filled Composite Columns used in C-SMF shall meet the following additional requirements:

- (1) The minimum required shear strength of the column shall meet the requirements in Part 6 Chapter 5.
- (2) The strong-column/weak-beam design requirements shall be met. Column Bases shall be designed to sustain inelastic flexural hinging.
- (3) The minimum wall thickness of concrete-filled rectangular HSS shall equal $b\sqrt{F_y/(2E_s)}$ for the flat width b of each face, where b is as defined in Part 6 Chapter 10 Table 10.2.1.

13.5.6 Composite Steel Plate Shear Walls (C-SPW)

13.5.6.1 Scope

This Section is applicable to structural walls consisting of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite Boundary Members. C-SPW shall meet the requirements of this section.

13.5.6.2 Wall Elements

13.5.6.2.1 Nominal Shear Strength

The nominal shear strength of C-SPW with a stiffened plate conforming to Section 13.5.4.2.2 shall be determined as:

$$V_{ns} = 0.6A_{sp}F_y \quad 13.5.3$$

where

V_{ns} = nominal shear strength of the steel plate, N

A_{sp} = horizontal area of stiffened steel plate, mm²

F_y = specified minimum yield strength of the plate, MPa

The nominal shear strength of C-SPW with a plate that does not meet the stiffening requirements in Section 13.5.4.2.2 shall be based upon the strength of the plate, excluding the strength of the reinforced concrete, and meet the requirements in the Part 6 Chapter 10, including the effects of buckling of the plate.

13.5.6.2.2 Detailing Requirements

The steel plate shall be adequately stiffened by encasement or attachment to the reinforced concrete if it can be demonstrated with an elastic plate buckling analysis that the composite wall can resist a nominal shear force equal to V_{ns} . The concrete thickness shall be a minimum of 102 mm (4 in.) on each side when concrete is provided on both sides of the steel plate and 200 mm (8 in.) when concrete is provided on one side of the steel plate. Headed shear stud connectors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet the detailing requirements in Part 6 Chapter 5. The reinforcement ratio in both directions shall not be less than 0.0025; the maximum spacing between bars shall not exceed 455 mm (18 in.).

The steel plate shall be continuously connected on all edges to structural steel framing and Boundary Members with welds and/or slip-critical high-strength bolts to develop the nominal shear strength of the plate. The Design Strength of welded and bolted connectors shall meet the additional requirements in Part 6 Chapter 10.