नेपाल राष्ट्रिय भवन संहिता

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NEPAL NATIONAL BUILDING CODE

NBC 111: 2025 (Proposed)

नेपालका स्टील भवनहरूका लागिअभ्यास संहिता

(प्रस्तावित)

CODE OF PRACTICE FOR STEEL BUILDINGS IN NEPAL

(Proposed)

नेपाल सरकार

शहरी विकास मन्त्रालय

सिंहदरबार, काठमाडौं, नेपाल

२०८१

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1 General

1.1 Title

1.1.1 "Nepal National Building Code NBC 111: Code of Practice for Steel Buildings in Nepal" is the title of this document. This document is the outcome of the revision of the earlier version of the NBC 111: 1994 Steel.

1.2 Scope

1.2.1 This publication is applicable to the design, fabrication and assembly of structural steel buildings and others structures exhibiting analogous functionality, characterized by comparable vertical and lateral load-bearing components specifically with hot rolled steel sections and high tensile strength steel. This publication does not apply to the following structures and materials:

i) Bridges
ii) Cranes
iii) Tanks
iv) Transmission towers
v) Materials less than 3 mm thick
vi) Cold-formed light gauge sections

For detailed information on loads to be considered, reference shall be made to NBC 103 for occupancy (imposed load), NBC 104 for Wind Load, NBC 105 for seismic load and NBC 106 for snow loads.

1.2.2 The requirements mentioned in this standard are merely the minimum necessary quality of materials, procedures and workmanships consistent with the assumption in design rules. The actual requirements might be more stringent and shall be further developed per project basis, the type of structure and the method of construction.

1.2.3 For detailed information on seismic design of steel structures, reference shall be made to NBC 105.

1.3 Referenced Specifications, Codes and Standards

The following are the standards referenced in this publication:

1. IS 8976 (1978): Guide for preparation and arrangement of sets of drawings and parts lists [PGD 24: Drawings]
2. IS 962 (1989): Code of practice for architectural and building drawings [CED 51: Planning, Housing and pre-fabricated construction]
3. IS 875 (Part 3): Wind loads on building structure
4. NBC 102: Unit Weight of Materials
5. NBC 103: Occupancy Load
6. NBC 104: Wind Load
7. NBC 105: Seismic Design of Buildings in Nepal
8. NBC 106: Snow Load
9. IS 800: 2007 General Construction in Steel – Code of Practice
10. NS 180 Dimensions for Hot rolled Steel Sections
    Part 1: Channel Sections
    Part 2: Angles
11. NS 295 Hot Rolled Steel Sections, Beam, Flat, Strip, Plate, Rectangular and Square Hollow
12. NS 427 Steel Tube for Structural use
13. IS 1363 Hexagon Head Bolts, Screws and Nuts of Product Grade C
14. IS 1364 Hexagon Head Bolts, Screws, And Nuts of Property Grades A And B
15. IS 3757 Specification for High Strength Structural Bolts
16. IS 4000 Code of practice for high strength bolts in steel structures
17. IS 5369 General Requirements for Plain Washers and Lock Washers

18. IS 5370 Specification for Plain Washers with Outside Diameter 3 X Inside Diameter
19. IS 5372 Specification for Taper Washers for Channels
20. IS 6610 Specification for Heavy Washers for Steel Structures
21. IS 6623 High Strength Structural Nuts
22. IS 6649 Specification for Hardened and Tempered Washers for High Strength Structural Bolts and Nuts
23. IS 4000 Code of practice for high strength bolts in steel structures
24. NS 157 Hexagonal Nut & Bolts
25. NS 202 Bolts, Screws & Studs - Nominal Length and Thread Lengths for General Purpose Bolts
26. AS 1163 Structural steel hollow sections
27. AS 1594 Hot-rolled steel flat products
28. AS/NZS 3678 Structural steel – Hot-rolled plates, floorplates and slabs
29. AS/NZS 3679 Structural steel
    Part 1 Hot-rolled bars and sections
    Part 2 Welded I-sections
30. IS 5624 Foundation Bolts Specification
31. IS 814 Covered Electrode for Manual Metal Arc Welding of Carbon and Carbon Manganese Steel
32. IS 1395 Low and Medium Alloy Steel Covered Electrodes for Manual Metal Arc Welding
33. IS 1278 Filler Rods and Wires for Gas Welding
34. IS 1387 General requirements for the supply of metallurgical materials
35. IS 15977 Classification and Acceptance Tests for Bare Solid Wire Electrodes and Wire Flux Combination for Submerged Arc Welding of Structural Steel - Specification
36. IS 6419 Welding rods and bare electrodes for gas shielded arc welding of structural steel
37. IS 6419 Welding rods and bare electrodes for gas shielded arc welding of structural steel
38. IS 6560 Welding Consumables- Wire Electrodes, Wires, Rods and Deposits for Gas Shielded Arc Welding of Creep-Resisting Steels- Classification
39. NS 151 Mild Steel for Metal Arc Welding Electrode Core Wire
40. AS 1163 Structural steel hollow sections
41. AS 1594 Hot-rolled steel flat products
42. AS/NZS 3678 Structural steel – Hot-rolled plates, floorplates and slabs
43. AS/NZS 3679 Structural steel
    Part 1 Hot-rolled bars and sections
    Part 2 Welded I-sections
44. IS 1030 Carbon steel castings for general engineering purposes
45. IS 2708 1.5% manganese steel castings for general engineering purposes
46. IS 875-2 (1987): Code of Practice for Design Loads (Other Than Earthquake) For Buildings And Structures, Part 2: Imposed Loads.

1.4 Definitions

Some terminologies related to design of steel structures used in this code are defined as follows:

1.4.1 **Accidental loads**: These are loads that arise due to low probability events such as vehicle collision, explosion, acts of terrorism, etc.

1.4.2 **Accompanying Load**: Live (imposed) load acting along with leading imposed load but causing lower actions and/or deflections.

1.4.3 **Action**: The primary cause for stress or deformations in a structure such as dead, live, wind, seismic or temperature loads.

1.4.4 **Anchor rod**: A mechanical device that is either cast in concrete or drilled and chemically adhered, grouted, or wedged into concrete and/or masonry for the purpose of the subsequent attachment of structural steel.

1.4.5 **Beam**: Nominally horizontal structural member that has the primary function of resisting bending moments.

1.4.6 **Beam-Column**: Structural member that resists both axial force and bending moments.

1.4.7 **Bearing Type Connection**: Bolted connection where shear forces are transmitted by the bolt bearing against the connection elements.

1.4.8 **Braced Member**: A member in which relative transverse displacement is effectively prevented by bracing.

1.4.9 **Block shear rupture**: In a connection, limit state of tension rupture along one path and shear yielding or shear rupture along another path.

1.4.10 **Brittle Cladding**: The load at which an element, a member or a structure as a whole, either collapses in service or buckles in a load test and develops excessive lateral (out of plane) deformation or instability.

1.4.11 **Buckling**: Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

1.4.12 **Buckling Strength or Resistance**: Force or moment, which a member can withstand without buckling.

1.4.13 **Built-up member**: Member fabricated from structural steel elements that are welded/ bolted together.

1.4.14 **Camber**: Curvature fabricated into a beam/ truss so as to compensate for deflection induced by loads.

1.4.15 **Capacity Design:** The design steps (beyond those in the basic design of non-yielding structural elements, members and connections), which consider the effects of inelasticity incurred in yielding members resulting in over-strength-based demands on the non-yielding structural elements.

1.4.16 **Capacity Protected Elements**: Components or members designed to remain elastic using capacity design principles when an adjacent component or member undergoes inelastic straining during design earthquake shaking.

1.4.17 **Characteristic Load (Action)**: The value of specified load (action), above which not more than a specified percentage (usually 5%) of samples of corresponding load are expected to be encountered.

1.4.18 **Characteristic Yield/Ultimate Stress**: The minimum value of stress, below which not more than a specified percentage (usually 5%) of corresponding stresses of samples tested are expected to occur.

1.4.19 **Cladding**: Exterior covering of structure.

1.4.20 **Collector**: A member that serves to transfer loads between diaphragms and the members of the vertical force-resisting elements of the lateral load resisting system.

1.4.21 **Column**: Members (generally vertical) resisting loads through axial, flexural and shearing actions.

1.4.22 **Column Base**: Assembly of columns, plates, connector weld or bolts, and anchor rods at the base of columns designed to transmit forces from the steel superstructure to reinforced concrete pedestal or foundation.

1.4.23 **Compact Section**: A cross-section, which can develop plastic moment, but has inadequate plastic rotation capacity needed for formation of a plastic collapse mechanism of the member or structure.

1.4.24 **Concentrically Braced Frame (CBF)**: A lateral load resisting system composed of interconnected beams and columns with inclined members as braces, which function as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resists gravity and lateral force effects primarily by axial actions.

1.4.24.1 **Special Concentrically Braced Frame (SCBF)**: A CBF specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

1.4.25 **Continuity Plate (CP)**: A plate provided within the flanges of the column at the levels of the flanges of the beams framing in the direction of the web of the column.

1.4.26 **Cover Plate**: Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus, or moment of inertia.

1.4.27 **Crane Load**: Horizontal and vertical loads from cranes.

1.4.28 **Dead Loads**: The self-weights of all permanent constructions and installations including the self-weight of all walls, partitions, floors, roofs, and other permanent fixtures acting on a member.

1.4.29 **Demand Critical Weld**: A weld connecting two structural components which are part of lateral load resisting system and at least one of which is expected to undergo inelastic straining following yielding.

1.4.30 **Design**: The process of establishing the physical and other properties of a structure for the purpose of achieving the desired strength, serviceability, durability, constructability, economy, and other desired characteristics. Design for strength, as used in this Specification, includes analysis to determine required strength and proportioning to have adequate available strength.

1.4.31 **Design Life**: Time period for which a structure or a structural element is required to perform its function without damage.

1.4.32 **Design Load/Factored Load**: A load value obtained by multiplying the characteristic load with a load factor.

1.4.33 **Diaphragm**: Roof, floor, or other membrane or bracing system that transfers in-plane forces to the lateral force-resisting system.

1.4.34 **Doubler Plate (DP)**: A plate provided parallel to the web of the column, and connected along its own perimeter to the web of the column and at some intermediate locations within itself when necessary.

1.4.35 **Drift**: Lateral deflection of structures.

1.4.36 **Ductility**: It is the property of the material or a structure indicating the extent to which it can deform beyond the limit of yield deformation before failure or fracture. The ratio of ultimate to yield deformation is usually termed as ductility.

1.4.37 **Durability**: It is the ability of a material to resist deterioration over long periods of time.

1.4.38 **Earthquake Loads**: The inertia forces produced in a structure due to the ground movement during an earthquake.

1.4.39 **Eccentrically Braced Frame (EBF)**: A lateral load resisting system composed of interconnected beams and columns with inclined members as braces that has at least one end connected to a beam through a link with a defined eccentricity from another beam-to-brace connection, with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resist gravity and lateral force effects primarily by axial action in the braces, and shearing and flexural actions in the links. It is specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

1.4.40 **Edge Distance**: Distance from the center of a fastener hole to the nearest edge of an element measured perpendicular to the direction of load transfer.

1.4.41 **Effective Length**: Length of an otherwise identical compression member with the same strength when analyzed with simple end conditions.

1.4.42 **Elastic Cladding**: Claddings, such as metal sheets, that can undergo considerable deformation without damage.

1.4.43 **Elastic Critical Moment**: The elastic moment, which initiates lateral-torsional buckling of a laterally unsupported beam.

1.4.44 **Elastic Analysis**: Structural analysis based on the assumption that the structure returns to its original geometry on removal of the load.

1.4.45 **Elastic Limit**: It is the stress below which the material regains its original size and shape when the load is removed. In steel design, it is taken as the yield stress.

1.4.46 **End Distance**: Distance from the center of a fastener hole to the edge of an element measured parallel to the direction of load transfer.

1.4.47 **End Return**: Length of fillet weld that continues around a corner in the same plane.

1.4.48 **Erection Loads**: The actions (loads and deformations) experienced by the structure exclusively during erection.

1.4.49 **Erection Tolerance**: Amount of deviation related to the plumbness, alignment, and level of the element as a whole in the erected position. The deviations are determined by considering the locations of the ends of the element.

Note: In the case of members with tire protection material applied, the exposed surface area is to be taken as the internal surface area of the fire protection material.

1.4.50 **Fastener**: Generic term for bolts, rivets, or other connecting devices.

1.4.51 **Fatigue**: Limit state of crack initiation and growth resulting from repeated application of live loads.

1.4.52 **Filler**: Plate used to build up the thickness of one component.

1.4.53 **Fire Resistance**: Property of assemblies that prevents or retards the passage of excessive heat, hot gases, or flames under conditions of use and enables the assemblies to continue to perform a stipulated function.

1.4.54 **First-order analysis**: Structural analysis in which equilibrium conditions are formulated on the un-deformed structure; second-order effects are neglected.

1.4.55 **Flexural Stiffness**: Stiffness of a member against rotation as evaluated by the value of bending deformation moment required to cause a unit rotation while all other degrees of freedom of the joints of the member except the rotated one are assumed to be restrained.

1.4.56 **Flexural Buckling**: Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

1.4.57 **Flexural-torsional Buckling**: Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

1.4.58 **Friction Type Connection**: Connection effected by using pre-tensioned high strength bolts where shear force transfer is due to mobilization of friction between the connected plates due to clamping force developed at the interface of connected plates by the bolt pre-tension.

1.4.59 **Fully restrained moment connection**: Connection capable of transferring moment with negligible rotation between connected members.

1.4.60 **Gauge**: Transverse center-to-center spacing of fasteners.

1.4.61 **Gusset Plate**: Plate element connecting truss members or a strut or brace to a beam or column.

1.4.62 **Imposed (Live) Load**: The load assumed to be produced by the intended use or occupancy including distributed, concentrated, impact, vibration and snow loads but excluding, wind, earthquake and temperature loads.

1.4.63 **Inelastic Analysis**: Structural analysis that takes into account inelastic material behavior, including plastic analysis.

1.4.64 **Instability**: Limit state reached in the loading of a structural component, frame, or structure in which a slight disturbance in the loads or geometry produces large displacements.

1.4.65 **Lacing**: Plate, angle, or other steel shape, in a lattice configuration, that connects two steel shapes.

1.4.66 **Limit State**: 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 have reached its ultimate load-carrying capacity (strength limit state).

1.4.67 **Link**: The segment of a beam that is located between the ends of the connections of two inclined braces in EBFs. The length of the link is defined as the clear distance between the ends of two diagonal braces.

1.4.68 **Load**: Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

1.4.69 **Member Imperfection**: Initial displacement of points along the length of individual members (between points of intersection of members) from their nominal locations, such as the out-of-straightness of members due to manufacturing and fabrication.

1.4.70 **Moment Resistant Frame (MRF)**: A lateral load resisting system composed of interconnected beams and columns, without structural walls and inclined members as braces, which function as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resists gravity and lateral force effects primarily by axial and flexural actions.

1.4.70.1 **Special Moment Resisting Frame (SMRF)**: A MRF specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

1.4.71 **Partial Safety Factor**: The factor normally greater than unity by which either the loads (actions) are multiplied or the resistances are divided to obtain the design values.

1.4.72 **Pitch**: Longitudinal center-to-center spacing of fasteners. Center-to-center spacing of bolt threads along axis of bolt.

1.4.73 **Plastic Analysis**: Structural analysis based on the assumption of rigid-plastic behavior, that is, that equilibrium is satisfied and the stress is at or below the yield stress throughout the structure.

1.4.74 **Plastic Collapse**: The failure stage at which sufficient number of plastic hinges have formed due to the loads (actions) in a structure leading to a failure mechanism.

1.4.75 **Plastic Hinge**: Fully yielded zone that forms in a structural member when the plastic moment is attained.

1.4.76 **Plastic Moment**: Theoretical resisting moment developed within a fully yielded cross section.

1.4.77 **Plastic Section**: Cross-section, which can develop a plastic hinge and sustain plastic moment over sufficient plastic rotation required for formation of plastic failure mechanism of the member or structure.

1.4.78 **Proof Stress**: The stress to which high strength friction grip (HSFG) bolts are pre-tensioned.

1.4.79 **Protected Zone**: Area of a member or connection element designed to undergo inelastic straining under design earthquake effects, and is required to be devoid of additional attachments or discontinuities resulting from fabrication and erection procedures.

1.4.80 **Prying Force**: Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt, and the reaction of the connected elements.

1.4.81 $\boldsymbol{P-\delta}$ **effect**: Effect of loads acting on the deflected shape of a member between joints or nodes.

1.4.82 $\boldsymbol{P-\Delta}$ **effect**: Effect of loads acting on the displaced location of nodes in a structure. In tiered building structures, this is the effect of loads acting on the laterally displaced location of floors and roofs.

1.4.83 **Rupture Strength**: Strength limited by breaking or tearing of members or connecting elements.

1.4.84 **Second-order effect**: Effect of loads acting on the deformed configuration of a structure; includes P-δ effect and P-Δ effect.

1.4.85 **Semi-compact Section**: Cross-section, which can attain the yield moment, but not the plastic moment before failure by plate buckling.

1.4.86 **Service Load**: Load under which serviceability limit states are evaluated.

1.4.87 **Serviceability Limit State**: Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, comfort of its occupants, or function of machinery, under typical usage.

1.4.88 **Shear Force**: The in-plane force at any transverse cross-section column or beam.

1.4.89 **Shear Lag**: Non-uniform tensile stress distribution in a member or connecting element in the vicinity of a connection.

1.4.90 **Shear Stress**: The stress component acting parallel to a face, plane or cross-section.

1.4.91 **Slender Section**: Cross-section in which the elements buckle locally before reaching yield moment.

1.4.92 **Slip**: In a bolted connection, limit state of relative motion of connected parts prior to the attainment of the available strength of the connection.

1.4.93 **Slot weld**: Weld made in an elongated hole fusing an element to another element.

1.4.94 **Snug Tight**: The tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard spanner.

1.4.95 **Stress**: Force per unit area caused by axial force, moment, shear, or torsion.

1.4.96 **Stress Concentration**: Localized stress considerably higher than average due to abrupt changes in geometry or localized loading.

1.4.97 **Structural Analysis**: Determination of load effects on members and connections based on principles of structural mechanics.

1.4.98 **Sway Member**: A Member in which the transverse displacement of one end of the member relative to the other is not effectively prevented.

1.4.99 **Tension field action**: Behavior of a panel under shear in which diagonal tensile forces develop in the web and compressive forces develop in the transverse stiffeners in a manner similar to a Pratt truss.

1.4.100 **Tensile Stress**: The characteristic stress corresponding to rupture in tension, specified for the grade of steel in the appropriate specification/standard.

1.4.101 **Test Load**: The factored load, equivalent to a specified load combination appropriate for the type of test being performed.

1.4.102 **Torsional Buckling**: Buckling mode in which a compression member twists about its shear center axis.

1.4.103 **Transverse**: Direction along the stronger axes of the cross-section of the member.

1.4.104 **Transverse Stiffener**: Web stiffener oriented perpendicular to the flanges, attached to the web.

1.4.105 **Ultimate Limit State**: The state which, if exceeded can cause collapse of a part or the whole of the structure.

1.4.106 **Unstiffened Element**: Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.

1.4.107 **Web Crippling**: Limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.

1.4.108 **Yielding**: Limit state of inelastic deformation that occurs when the yield stress is reached.

1.4.109 **Yield Stress**: The characteristic stress of the material in tension before the elastic limit of the material is exceeded, as specified in the specification/standard.

1.5 Notations

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

- A -Area of cross-section
- $\mathrm{A_g}$ -Gross area of cross-section
- $\mathrm{A_e}$ -Effective area of cross-section
- $\mathrm{A_f}$ -Area of Flange
- A_g - Gross cross-section area
- A_{gf} -Gross cross-sectional area of outstanding (not- corrected) leg of a member
- A_n - Net area of the cross-sectional area of bolt
- A_{no} - Net cross-sectional area of outstanding (not-connected) leg of a member
- A_{pb} - Nominal bearing area of bolt on any plate
- A_q - Cross-sectional area of a bearing (load carrying) stiffner in contact with the flange
- A_s - Tensile stress area
- A_{sb} - Gross cross-sectional area of a bolt at the shank
- A_{tg} - Gross sectional area in tension from the centre of the hole to the toe of the angle section, etc perpendicular to the line of force
- A_{tn} - Net sectional area in tension from the center of the hole to the toe of the angle perpendicular to the line of force
- A_v - Shear area
- A_{vg} - Gross cross-sectional area in shear along the line of transmitted force
- a, b - Large cross-sectional area in shear along the line of transmitted force
- a_o - Peak acceleration
- a_1 - Unsupported length of individual of individual elements being laced between lacing points
- B - Length of sides of cap or base plate of a column
- b - Outstand/width of the element
- b_1 - Stiff bearing length, stiffener bearing length
- b_e - Effective width of flange between pair of bolts
- b_f - width of the flange
- b_i - width of flange as an internal element
- b_o - width of flange outside
- b_p - Panel zone width between column flanges at beam-column junction
- b_s - Shear lag distance
- b_t - Width of tension field
- b_w - Width of outstanding leg
- C - Center- to center longitudinal distance of battens
- C_m - Coefficient of thermal expansion
- C_{my}, C_{mz} - moment amplification factor about the respective axis
- c -Spacing of transverse stiffeners
- c_b - Moment amplification factor for braced member
- c_m - Moment reduction factor for lateral torsional bucking strength calculation
- c_s - Moment amplification factor for sway frame
- D - Overall depth/ diameter of the cross-sectional section
- d - Depth of web, nominal diameter
- d_2 - Twice the clear distance frpm the compression flange angles, plates or tongue plates to the neutral axis
- d_h - Diameter of a bolt hole
- d_o - Nominal diameter of the pipe column or the dimensions of the column in the depth direction of the base plate
- d_p - Panel zone depth in the beam-column junction
- $E(T)$ - Modulus of elasticity for steel at $T°C$
- $E(20)$ - Modulus of elasticity of the panel material Buckling strength
- E_p - Modulus of elasticity of the panel materials
- F_{cdw} - Buckling strength of un-stiffened beam web under concentrated load
- F_d - Factored design load
- F_n - Normal force
- F_o - Minimum proof pretension in high strength friction grip bolts
- F_{psd} - Bearing capacity of load carrying stiffener
- F_q - Stiffener force
- F_{qd} - Stiffener buckling resistance
- F_{test} - Test load
- $F_{test,a}$ - Load for acceptance test

$F_{test,min}$ - Minimum test load from the test to failure
$F_{test,S}$ -Strength test load
F_w - Design capacity of the web in bearing
F_x - External load, force or reaction
F_{xd} - Buckling resistance of load carrying web stiffener
f - Actual normal stress range for the detail category
f_1 - Frequency for a simply supported one way system
f_2 - Frequency of floor supported on steel girder perpendicular to the joist
f_a - Calculated stress due to axial force at service load
f_{abc} - Permissible bending stress compression at service load
f_{ac} - Permissible compressive in stress at service load
f_{abt} - Permissible bending stress in tension at service load
f_{apb} - Permissible bearing stress of the bolt at service load
f_{asb} - Permissible stress of the bolt in shear at service load
f_{at} - Permissible tensile stress at service load
f_{atb} - Permissible tensile stress of the bolt it service load
f_{aw} - Permissible stress of the weld service load
f_b - Actual bending stress at service
f_{bc} - Actual bending stress in compression at service load
f_{bd} - Design bending compressive stress corresponding to lateral buckling
f_{br} - Actual bearing stress due to bending at service load
f_{bt} - Actual bending stress in tension at service load
f_{bs} - Actual stress of weld at service load
f_{cc} - Elastic buckling stress of a column, Euler buckling stress
f_{cd} - Design compressive stress
$f_{cr,b}$ - Extreme fiber compressive stress corresponding elastic lateral buckling moment
f_e - Equivalent stress at service load
f_f - Fatigue stress range corresponding to 5×10^6 cycles of loading
f_{feq} - Equivalent constant amplitude stress
f_{fd} - Design normal fatigue strength
$f_{f,Max}$ - Highest normal stress range
f_{fn} - Normal fatigue stress range
f_{nw} - Normal stress in weld at service load
f_o - Proof stress
f_p - Actual bearing stress at service load
f_{pb} - Actual bearing stress in bending at service load
f_{psd} - Bearing strength of the stiffeners
f_r - Frequency
f_{sb} - Actual shear stress in bolt at service load
f_t - Actual tensile stress at service load
f_{tb} - Actual tensile stress of the bolt at service load
f_u - Characteristic ultimate tensile stress
f_{ub} - Characteristic ultimate tensile stress of the bolt
f_{um} - Average ultimate stress of the material as obtained from test
f_{up} - Characteristic ultimate tensile stress of the connected plate
f_v - Applied shear stress in the panel designed utilizing tension field action
f_w - Actual stress of weld at service load
f_{wd} - Design stress of weld at service load
f_{wn} - Nominal strength of fillet weld
f_x - Maximum longitudinal stress under combined axial force and bending
f_y - Characteristic yield stress
$f_y(T)$ - Yield stress of steel at $T°C$
$f_y(20)$ - Yield stress of steel at 20°C
f_{yb} - Characteristic yield stress of bolt
f_{yf} - Characteristic yield stress of flange
f_{ym} - Average yield stress as obtained from test
f_{yp} - Characteristic yield stress of connected plate
f_{yq} - Characteristic yield stress of stiffener material
G - Modulus of rigidity for steel
g - Gauge length between centre of the holes perpendicular to the load direction, acceleration due to gravity
h - Depth of the section
h_b - Total height from the base to the floor level concerned
h_c - Height of the column
h_e - Effective thickness
h_f - Center to center distance of flanges
h_i - Thickness of fire protection material
h_L - Height of the lip
h_s - Storey height
h_y - Distance between shear center of the two flanges of a cross-section

I - Moment of inertia of the member about an axis perpendicular to the plane of the frame

I_{fc} - Moment of inertia of the compression flange of the beam about the axis parallel to the web

I_{ft} - Moment of inertia of the tension flange of the beam about minor axis

I_q - Moment of inertia of a pair of stiffener about the centre of the web, or a single stiffener about the face of the web

I_s - Second moment of inertia

I_{so} - Second moment of inertia of the stiffener about the face of the element perpendicular to the web

I_T - Transformed moment of inertia of the one way system (in terms of equivalent steel, assuming the concrete flange of width equal to the spacing of the beam to be effective)

I_t - St. Venant’s torsion constant

I_w - Warping constant

I_y - Moment of inertia about the minor axis of the cross-section

I_Z - Moment of inertia about the major axis of the cross-section

K_b - Effective stiffness of the beam and column

K_h - Reduction factor to account for the high strength friction grip connection, bolts in over-sized and slotted holes

KL - Effective length of the member

KL/r - Appropriate effective slenderness ratio of the section

KL/r_y - Effective slenderness ratio of the section about the minor axis of the section

KL/r_z - Effective slenderness ratio of the section about the major axis of the section

$\left(\frac{KL}{r}\right)_o$ - Actual maximum effective slenderness ratio of the laced column

$\left(\frac{KL}{r}\right)_e$ - Effective slenderness ratio of the laced column accounting for shear deformation

K_v - Shear buckling co-efficient

K_w - Warping restraint factor

k - Regression coefficient

k_{sm} - Exposed surface area to mass ratio

L - Actual length, unsupported length, Length centre-to-centre distance of the intersecting members, cantilever length

L_c - Length of end connection in bolted and welded members, taken as the distance between outermost fasteners in the end connection, or the length of the end weld, measured along the length of the member

L_{LT} - Effective length for lateral torsional buckling

L_m - Maximum distance from the restraint for the compression flange at the plastic hinge to an adjacent restraint (Limiting distance)

L_o - Length between points of zero ‘moment (inflection) in the span

l - Centre-to-centre length of the supporting member

l_e - Distance between prying force and bolt centre line

l_g - Grip length of bolts in a connection

l_j - Length of the joint

l_s - Length between points of lateral support to the compression flange in a beam

l_v - Distance from bolt centre line to the toe of fillet weld or to half the root radius for a rolled section

l_w - Length of weld

M - Bending moment

M_a - Applied bending moment

M_{cr} - Elastic critical moment corresponding to lateral torsional buckling of the beam.

M_d - Design flexural strength

M_{dv} - Moment capacity of the section under high shear

M_{dy} - Design bending strength about the minor axis of the cross-section

M_{dx} - Design bending strength about the major axis of the cross-section

M_{eff} - Reduced effective moment

M_{fr} - Reduced plastic moment capacity of the flange plate

M_{fd} - Design plastic resistance of the flange alone

M_{nd} - Design bending strength under combined axial force and uniaxial moment

M_{ndy}, M_{ndx} - Design bending strength under combined axial force and the respective uniaxial moment acting alone

M_p -Plastic moment capacity of the section

M_{pb} - Moment in the beam at the intersection of the beam and column centre lines

M_{pc} - Moments in the column above and below the beam surfaces

M_{pd} - Plastic design strength

M_{pdf} - Plastic design strength of flanges only

M_q - Applied moment on the stiffener

M_s - Moment at service (working) load

M_{tf} - Moment resistance of tension flange

M_y - Factored applied moment about the minor axis of the cross-section

M_{yq} - Moment capacity of the stiffener based on its elastic modulus

M_x - Factored applied moment about the major axis of the cross-section

N - Number of parallel planes of battens

N_d - Design strength in tension or in compression

N_f - Axial force in the flange

N_{SC} - Number of stress cycles

n - Number of bolts in the bolt group/ critical section

n_e - Number of effective interfaces offering frictional resistance to slip

n_n - Number of shear planes with the threads intercepting the shear plane in the bolted connection

n_s - Number of shear planes without threads intercepting the shear plane in the bolted connection

P - Factored applied axial force

P_{cc} - Elastic buckling load

P_d - Design axial compressive strength

P_{dy}, P_{dz} - Design compression strength as governed by flexural buckling about the respective axis

P_e - Elastic Euler buckling load

P_{min} - Minimum required strength for each flange splice

P_r - Required compressive strength

P_s - Actual compression at service load

P_y - Yield strength of the cross-section under axial compression

p - Pitch length between centres of holes parallel to the direction of the load

p_s - Staggered pitch length along the direction of the load between lines of the bolt holes (see Figure 8)

Q - Prying force

Q_a - Accidental load (Action)

Q_c - Characteristic loads (Action)

Q_d - Design load (Action)

Q_p - Permanent loads (Action)

Q_v - Variable loads (Action)

q - Shear stress at service load

R - Ratio of the mean compressive stress in the web (equal to stress at mid depth) to yield stress of the web: reaction of the beam at support

R_d - Design strength of the member at room temperature

R_i - Net shear in bolt group at bolt "i"

R_r - Response reduction factor

R_{tf} - Flange shear resistance

R_u - Ultimate strength of the member at room temperature

r - Appropriate radius of gyration

r_l - Minimum radius of gyration of the individual element being laced together

r_f - Ratio of the design action on the member under fire to the design capacity

r_{vv} - Radius of gyration about the minor axis (v-v) of angle section

r_y - Radius of gyration about the minor axis

r_x - Radius of gyration about the major axis

S - Minimum transverse distance between the centroid of the bolt group or weld group

S_c - Constant stress range

S_d - Design strength

S_o - Original cross-sectional area of the test specimen

S_p - Spring stiffness

S_u - Ultimate strength

s_c - Anchorage length of tension field along the compression flange

s_t - Anchorage length of tension field along the tension flange

s_a - Actual stiffener spacing

T - Temperature in degree Celsius: Factored tension

T_b - Applied tension in bolt

T_{cf} - Thickness of compression flange

T_d - Design strength under axial tension

T_{dg} - Yielding strength of gross section under axial tension

T_{dn} - Rupture strength of net section under axial tension

T_{db} - Design strength of bolt under axial tension; Block shear strength at end connection

T_e - Externally applied tension

T_f - Factored tension force of friction type bolt

T_l - Limiting temperature of the steel

T_{nb} - Nominal strength of bolt under axial tension

T_{nd} - Design tension capacity

T_{ndf} - Design tension capacity of friction type bolt

T_{nf} - Nominal tensile strength of friction type bolt

T_s - Actual tension under service load

t - Thickness of element/angle, time in minutes

t_f - Thickness of flange

t_p - Thickness of plate

t_{pk} - Thickness of packing

t_q - Thickness of stiffener

t_s - Thickness of base slab

t_t - Effective throat thickness of welds

t_w - Thickness of web

V - Factored applied shear force

V_b - Shear in batten plate

V_{bf} - Factored frictional shear force in friction type connection

V_{cr} - Critical shear strength corresponding to web buckling

V_d - Design shear strength

V_{db} - Block shear strength

V_{nb} - Nominal shear strength of bolt

V_{nbf} - Bearing capacity of bolt for friction type connection

V_p - Plastic shear resistance under pure shear

V_n - Nominal shear strength

V_{npb} - Nominal bearing strength of bolt

V_{nsf} - Nominal shear capacity of a bolt Nominal shear capacity of bolt as governed by slip in friction type connection

V_s - Transverse shear at service load

V_{sb} - Factored shear force in the bolt

V_{sd} - Design shear capacity

V_{sdf} - Design shear strength in friction type bolt

V_{sf} - Factored design shear force of friction bolts

V_t - Applied transverse shear

V_{tf} - Shear resistance in tension field

W - Total load

w - Uniform pressure from below on the slab base due to axial compression under the factored load

w_{tf} - Width of tension field

x_t - Torsional index

Z_e - Elastic section modulus

Z_{ec} - Elastic section modulus of member with respect to extreme compression fiber

Z_{et} - Elastic section modulus of the member with respect to extreme tension fiber

Z_p - Plastic section modulus

Z_v - Contribution to the plastic section modulus of the total shear area of the cross-section

y_g - Distance between point of application of the load and shear center of the cross-section

y_s - Co-ordinate of the shear center in respect to centroid

α - Imperfection factor for buckling strength in columns and beams

α_t - Coefficient of thermal expansion

β_M - Ratio of smaller to the larger bending moment at the ends of a beam column

β_{My}, β_{Mx} - Equivalent uniform moment factor for flexural buckling for y-y and z-z axes respectively

β_{MLT} - Equivalent uniform moment factor for lateral torsional buckling

χ - Strength reduction factor to account for buckling under compression

χ_m - Strength reduction factor, χ, at f_{ym}

χ_{LT} - Strength reduction factor to account for lateral torsional buckling of beams

δ - Storey deflection

δ_L - Horizontal deflection of the bottom of storey due to combined gravity and notional load

δ_p - Load amplification factor

δ_U - Horizontal deflection of the top of storey due to combined gravity and notional load

ϕ - Inclination of the tension field stress in web

γ - Unit weight of steel

γ_f - Partial safety factor for load

γ_m - Partial safety factor for material

γ_{m0} - Partial safety factor against yield stress and buckling

γ_{m1} - Partial safety factor against ultimate stress

γ_{mb} - Partial safety factor for bolted connection with bearing type bolts

γ_{mf} - Partial safety connection factor for bolted with High Strength Friction Grip bolts

γ_{fft} - Partial safety factor for fatigue load

γ_{mft} - Partial safety factor for fatigue strength

γ_{mv} - Partial safety factor against shear failure

γ_{mw} - Partial safety factor for strength of weld

ϵ - Yield stress ratio $\left(\frac{230}{f_y}\right)^{\frac{1}{2}}$

λ - Non-dimensional slenderness ratio

$$= \sqrt{f_y \left(\frac{KL}{r}\right)^2 / \pi^2 E} = \sqrt{f_y / f_{cc}}$$

$$= \sqrt{P_y / P_{cc}}$$

λ_{cr} - Elastic buckling load factor

λ_e - Equivalent slenderness ratio

λ_{LT} - Non-dimensional slenderness ratio in lateral bending

λ_{scr} - Elastic buckling load factor of each storey

μ - Poisson's ratio

μ_c - Correction factor

μ_f - Coefficient of friction (slip factor)

μ_r - Capacity reduction factor

θ - Ratio of the rotation at the hinge point to the relative elastic rotation of the far end of the beam segment containing plastic hinge

ρ - Unit mass of steel

τ - Actual shear stress range for the detail category

τ_b - Buckling shear stress

τ_{ab} - Permissible shear stress at the service load

$\tau_{cr,e}$ - Elastic critical shear stress

τ_f - Fatigue shear stress range

$\tau_{f,Max}$ - Highest shear stress range

τ_{fd} - Design shear fatigue strength

τ_{fn} - Fatigue shear stress range at N_{SC} cycle for the detail category

τ_v - Actual shear stress at service load

ψ - Ratio of the moments at the ends of the laterally unsupported length of a beam

Γ - Frame buckling load factor

NOTE—The subscripts y, x denote the y-y and x-x axes of the section, respectively. For symmetrical sections, y-y denotes the minor principal axis whilst x-x denotes the major principal axis (see 1.8.3).

1.6 Units

Unless otherwise noted, this code uses the system of units in SI system namely, kilogram, meter, second, Pascal and Newton (kg, m, s, Pa and N) for mass, time, stress and force (load) respectively.

1.7 Tolerances

All the steel products conforming to this standard shall be those specified in IS 1852 except for parallel flange beams and columns covered by IS 12778 for which the tolerances shall be as per IS 12779. Other tolerances may be followed within the total tolerance range as specified in IS 1852 and IS 12779 as applicable.

1.8 Structural Design Documents and Conventions

The design documents and specifications provided for construction shall be drafted so that they are easy to read and shall be drawn to a scale that makes the information clear.

1.8.1 Drawings

Reference shall be made to IS 8976 and IS 962 for drawings and stress sheet.

1.8.1.1 Plans

The plans shall be clear such that the necessary dimensions and location of the structural members are easily understandable. Story heights, center lines, offsets shall be clearly shown and dimensioned. Any specific instruction from the designer to be considered during construction shall be clearly mentioned not excluding the type of construction and the type of connection. Any further information if need on the assumed loads is also to be written.

1.8.2 Shop Drawings

Shop drawings shall contain sufficient information not excluding the type, dimension, relative location and detail of connections, to ensure convenient assembly and erection at site and shall be prepared well in advance of actual fabrication. All shop drawings shall confirm to relevant code.

1.8.3 **Axes convention**

The normal convention for specifying axes, as shown in Figure 1 , shall be as follows:

a) Z-Z along member length

b) X-X along cross-section
Parallel to the flanges
Parallel to the shorter leg in case of angle sections

c) Y-Y along cross-section
Perpendicular to the flanges
Perpendicular to the shorter leg in case of angle sections

d) U-U major axis (when the X-X axis is not the same)

e) V-V minor axis (when the Y-Y axis is not the same)

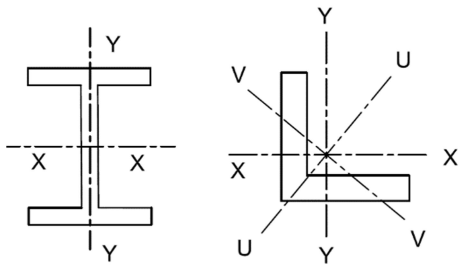


Figure 1: Axes Convention

2 Materials

2.1 General

2.1.1 The material properties given in this section are nominal values, to be accepted as characteristic values in design calculations.

2.1.2 Fundamentals of material physical properties

Regarding basic physical properties, the materials used in steel structures must satisfy the following requirements.

a) Adequate strength and deformability, or toughness
b) Resistance to change or deterioration in quality during the service life
c) Minimum impact on the human beings and environment

2.2 Structural Steel

2.2.1 The dimensions, form, weight, tolerances of all rolled shapes shall conform to the following code:

a) Indian Standards

IS 806	Code of Practice for Use of Steel Tubes in General Building Construction
IS 808	Hot Rolled Steel Beam Column Channel and Angle Sections - Dimensions and Properties
IS 1173	Specification for Hot Rolled and Slit Steel Tee Bars
IS 1239	Steel Tubes, Tubulars and Other Wrought Steel Fittings - Part 1: Steel Tubes
IS 1730	Steel Plates, Sheets, Strips and Flats for Structural and General Engineering Purpose
IS 1732	Steel Bars, Round and Square for Structural and General Engineering Purposes
IS 3954	Hot Rolled Steel Channel Sections for General Engineering Purposes-Dimensions

b) Nepal Standards

NS 180	Dimensions for Hot rolled Steel Sections Part 1: Channel Sections Part 2: Angles
NS 295	Hot Rolled Steel Sections, Beam, Flat, Strip, Plate, Rectangular and Square Hollow
NS 427	Steel Tube for Structural use

Note: If structural steel or shapes other than those refereed in a and b are used, they shall comply with a standard approved by the design engineer.

2.2.2 Acceptance of Steels

Certified mill test reports, or test certificates issued by the mill, shall constitute sufficient evidence of compliance with the material supply standards referred to in this Standard.

Other materials used in association with structural steel work shall conform appropriate standards.

2.2.3 Structural steel other than those specified in 2.2.1 are suitably modified and the steel is also suitable for the type of fabrication adopted.

2.2.4 Steel that is not supported by mill test results may be used only in unimportant members and details, where their properties such as ductility and weldability would not affect the performance requirements of the members and the structure as a whole.

However, such steels may be used in structural system after confirming their qualities by carrying appropriate tests in accordance with the method specified in IS 1608.

2.2.5 Properties

2.2.5.1 The material coefficients to be adopted in calculations for the structural steels covered by this code shall be taken as follows:

a) Unit mass of steel. $\rho = 7850\ \mathrm{kg/m^3}$
b) Modulus of elasticity, $\mathrm{E} = 2.0 \times 10^5\ \ \mathrm{N/mm^2}\ \ \ (\mathrm{MPa})$
c) Poisson ratio, $\mu = 0.30$
d) Modulus of rigidity (Shear modulus), $\mathrm{G} = 0.769 \times 10^5\ \ \mathrm{N/mm^2}\ \ \ (\mathrm{MPa})$
e) Co-efficient of thermal expansion, $\alpha_t = 12 \times 10^{-6}$ per °C

2.2.5.2 Mechanical Properties of Structural Steel

The key mechanical properties of structural steel that are critical for design include yield stress (f_y), tensile or ultimate stress (f_u), maximum percentage elongation over a standard gauge length, and notch toughness. All of these properties, except for notch toughness, are determined through tensile testing of specimens taken from plates, sections, and similar elements, following the relevant standards.

2.3 Bolts, Nuts and Washers

Bolts, nuts and washers shall be in accordance with:

a) Indian Standards

IS 1363	Hexagon Head Bolts, Screws and Nuts of Product Grade C
IS 1364	Hexagon Head Bolts, Screws, And Nuts of Property Grades A And B
IS 3757	Specification for High Strength Structural Bolts
IS 4000	Code of practice for high strength bolts in steel structures
IS 5369	General Requirements for Plain Washers and Lock Washers
IS 5370	Specification for Plain Washers with Outside Diameter 3 X Inside Diameter
IS 5372	Specification for Taper Washers for Channels
IS 6610	Specification for Heavy Washers for Steel Structures
IS 6623	High Strength Structural Nuts
IS 6649	Specification for Hardened and Tempered Washers for High Strength Bolts and Nuts
IS 4000	Code of practice for high strength bolts in steel structures

b) Nepal Standards

NS 157	Hexagonal Nut & Bolts
NS 202	Bolts, Screws & Studs - Nominal Length and Thread Lengths for General Bolts

2.4 Anchor Rods and Threaded Rods

Anchor bolts shall be manufactured from rod complying with steel standards of provided that the thread comply with the following codes.

a) American Standards

ASTM F1554	Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength
ASTM A307	Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rods 60,000 PSI Tensile Strength

b) Indian standard

IS 1367 Technical Supply Conditions for Threaded Steel Fasteners
IS 5624 Foundation Bolts - Specification

2.5 Mechanical and Chemical Anchors

All mechanical and chemical fasteners shall be designed and specified in the construction specification in accordance with:

a) American Standards

ACI 355.2 Qualification of Post-Installed Mechanical Anchors in Concrete
ACI 355.4 Qualification of Post-Installed Adhesive Anchors in Concrete
ASTM E488/E488M Standard Test Methods for Strength of Anchors in Concrete Elements
ASTM E1512 Standard Test Methods for Testing Bond Performance of Bonded Anchors

2.6 Welding Consumable

2.6.1 All welding consumables and deposited weld metal shall be in accordance with:

a) Indian Standards

IS 814	Covered Electrode for Manual Metal Arc Welding of Carbon and Carbon Manganese Steel
IS 1395	Low and Medium Alloy Steel Covered Electrodes for Manual Metal Arc Welding
IS 1278	Filler Rods and Wires for Gas Welding
IS 1387	General Requirements for the Supply of Metallurgical Materials
IS 15977	Classification and Acceptance Tests for Bare Solid Wire Electrodes and Wire Flux Combination for Submerged Arc Welding of Structural Steel - Specification
IS 6419	Welding Rods and Bare Electrodes for Gas Shielded Arc Welding of Structural Steel
IS 6419	Welding Rods and Bare Electrodes for Gas Shielded Arc Welding of Structural Steel
IS 6560	Welding Consumables- Wire Electrodes, Wires, Rods and Deposits for Gas Shielded Arc Welding of Creep-Resisting Steels- Classification

b) Nepal Standards

NS 151 Mild Steel for Metal Arc Welding Electrode Core Wire

2.7 Steel Casting

All steel casting shall be in accordance with:

IS 1030	Carbon Steel Castings for General Engineering Purposes
IS 2708	1.5 Percent Manganese Steel Castings for General Engineering Purposes

2.8 Other materials

Other materials used in association with structural steel work shall conform to relevant standards and approval of the design engineer and engineer-in-charge.

3 General Design Requirements

3.1 Aim

3.1.1 The overarching aim of structural steel design is to result a structure which has sufficient stability and strength and satisfies criteria imposed by serviceability.

3.1.2 A structure is said stable if it has adequate resistance to overturning, sliding or lifting.

3.1.3 A structure is strong if it has adequate resistance against structural failures.

3.1.4 A structure is serviceable if it has adequate resistance against loss of serviceability

3.2 Design Basis

3.2.1 The design of steel structures shall be based on limit state method. The resulting design shall have sufficient stability, strength and shall be serviceable under all possible load combinations.

3.2.2 In cases, when it is established that limit state design cannot be embraced pragmatically, working stress method is also permitted according to Annex B: Working Stress Design.

3.3 Loads and Load Combinations

3.3.1 The following loads and their associated load effects if applicable shall be considered along with their appropriate partial safety factors for steel design:

a) Dead Loads
b) Imposed loads
c) Lateral loads (Earthquake or wind)
d) Erection loads
e) Accidental loads such as due to blast, impact of vehicles, etc.
f) Secondary effects (geometry changes due to temperature variation, differential settlements, imperfections in erection, eccentric connections, rigidity of joints differing from design assumptions.)

3.3.2 NBC 102 shall be referred to for ascertaining the dead load of the structure.

3.3.3 NBC 103 shall be referred to for ascertaining the imposed load.

3.3.4 IS 975 (Part 3) shall be referred to for ascertaining the wind load.

3.3.5 NBC 105 shall be referred to for ascertaining the seismic load.

3.3.6 NBC 106 shall be referred to for ascertaining the snow load.

3.3.7 The consideration of wind and earthquake shall not be simultaneous in design.

3.3.8 Loads during Erection

The parts of a structure and the structure as a whole shall be capable of sustaining the loads arising during Erection phase. These erection loads comprise of the summation of dead load, imposed load and wind load.

3.3.9 Thermal Effects

The variation of temperature shall be considered in design. The absolute maximum and minimum temperature in different parts of Nepal may be obtained from the Department of Hydrology and Metrology and the same shall be considered while accounting for expansion and contractions in steel.

The effect of differential temperature variations between material and air and within material itself due to part exposure to sunlight shall also be considered.

3.3.10 Load Combinations

Load Combinations taken for design purposes shall correspond that combination of loads that produce the maximum stresses and deformation in the structure to be designed. The following combinations supplemented by partial safety factors as in Table 3: Load factors, γ_i for various load types and combinationsmay be taken into account.

a) Dead Load + Imposed Load
b) Dead Load + Imposed Load + Lateral Load
c) Dead Load + Lateral Load
d) Dead Load + Erection Load

In case the structure supports cranes, the imposed load shall include the crane effects.

3.4 Crane Load consideration

3.4.1 Crane load effects on structure includes vertical loading, effects by eccentric vertical loadings, impact factors, lateral loading and longitudinal loadings however not all of them acting simultaneously across and along the crane rail.

3.4.2 The effects of crane on the structure as listed in the above clause shall be accounted in design as per the provision of IS 875 part 2.

3.4.3 The load combinations including the crane loads shall be accounted for in consultation with the client. The following correspond to the minimum combinations to be accounted for in case of any specific direction:

a) Static vertical load of all cranes in operation along with impact loading and horizontal thrust from one of the cranes in a way that the effect on the structure is maximized. In case of tandem operation, the impact loading and horizontal thrust shall be considered from both of the cranes.

b) In case when multiple cranes span multiple bays of the structure, the load shall be taken as subjected to the above clause with the cranes being positioned in two of the longest bays of the structure.

c) Longitudinal loading for a minimum of two loaded crane tracks shall be considered in the load combinations.

3.4.4 The effect of cranes shall be taken into account while analyzing for the effect of earthquake such that worst possible combination of crane effects is being considered

3.5 Section Properties

3.5.1 Determination of Gross Area and Net Area

3.5.1.1 The gross area A_g of a member is the total cross—sectional area of the section used without deduction for bolt holes.

3.5.1.2 The net area of A_n of a member is the area deducted after making due consideration for the presence of bolt holes in the cross-section.

3.5.1.3 Holes shall be deducted in excess of 3 mm of the actual diameter for calculating net areas for all members except in compression.

3.5.2 Cross sectional limit states for local buckling

In case of compression, the local buckling of parts of a compression element may prevent the element from bearing the overall load it was designed for. In order to prevent this, the capacity of each element comprising the cross-section shall be greater than their combination. Thus, in order to prevent this undesired circumstance, four classes of sections are recognized:

a) Class 1 (Plastic)
These are the desired sections for the plastic design of structures. They allow a section to attain its full plastic capacity with hinge rotation without local buckling. Width to thickness ratio limits of elements of a cross-section deemed as plastic shall satisfy those given in Table 1 under the category Plastic.

b) Class 2 (Compact)

Compact sections allow element to develop plastic resistance but do not have sufficient capacity for hinge rotation due to plastic mechanism. The width to thickness ratio limits shall satisfy those in Table 1 for category Compact.

c) Class 3 (Semi-compact)

Semi-compact sections do not have adequate capacity for the entirety of the section to reach plastic resistance. Only the extreme fibers reach plastic resistance capacity before the onset of local buckling. The width to thickness ratio limits shall satisfy those given in Table 1 under the category Semi-compact.

d) Class 4 (Slender)

These are sections in which the onset of local buckling is met before any part of the section reaches plastic resistance capacity. Hence, it is desirable to make explicit allowance for local buckling. The sections not falling under the limits of category Semi-compact as given in Table 1 are deemed as slender sections.

3.5.2.1 When different elements comprising a cross-section fall under different classes as listed in Table 1, the cross-section shall be classified such that it corresponds to the plastic resistance capacity of the element most susceptible to local buckling.

3.5.3 Types of elements:

a) Unstiffened elements

These are elements supported along just one edge parallel to the direction of the compression load. Examples are shown Figure 2.

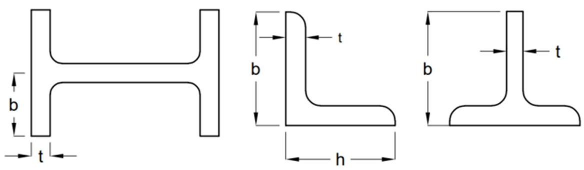


Figure 2: Unstiffened elements

b) Stiffened elements

These are elements supported long two edges parallel to the direction of the compression force. Examples are shown in Figure 3.

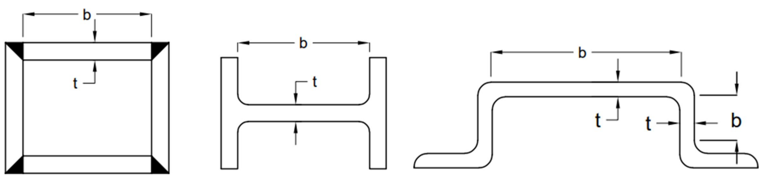


Figure 3: Stiffened elements

Type of Element	**Compression component**		**Ratio**	**Class of section**		
				Class 1	**Class 2**	**Class 3**
Unstiffened Element	Outstanding element of compression flange	Rolled section	b/t_f	9.4ϵ	10.5ϵ	15.7ϵ
		Welded section	b/t_f	8.4ϵ	9.4ϵ	13.6ϵ
	Single angle, or double angles with the components separated, pure compression (must fulfill all three limits)		b/t d/t $(b+d)/t$	$42\ \epsilon$		15.7ϵ 15.7ϵ 25ϵ
	Angle in bending		b/t d/t	9.4ϵ 9.4ϵ	10.5ϵ 10.5ϵ	15.7ϵ 15.7ϵ
	Outstanding leg of an angle 1. in contact back-to-back in a double angle manner 2. with its back in continuous contact with another component		d/t	9.4ϵ	10.5ϵ	15.7ϵ
	Stem of a T-section		d/t	8.4ϵ	9.4ϵ	18.9ϵ
Stiffened Element	Internal element of compression flange	Flexural compression	b/t_f	29.3ϵ	33.5ϵ	42ϵ
		Axial compression	b/t_f	$42\ \epsilon$		42ϵ
	Web of a channel		d/t_w	42ϵ	42ϵ	42ϵ
	Web of an I, H or box section	Neutral axis at mid-depth	d/t_w	84ϵ	105ϵ	126ϵ
		Generally, If r_1 is negative	d/t_w	$\frac{84\epsilon}{1+r_1}$ but $\geq 42\epsilon$	$\frac{105\epsilon}{1+r_1}$	$\frac{126\epsilon}{1+2r_1}$ but $\geq 42\epsilon$
		Generally, If r_1 is positive	d/t_w		$\frac{105\epsilon}{1+1.5r_1}$ but $\geq 42\epsilon$	
		Axial compression	d/t_w	Not Applicable		42ϵ
	Circular hollow tube subjected to: 1. Moment		D/t	$42\epsilon^2$	$52\epsilon^2$	$146\epsilon^2$
	2. Axial Compression		D/t	Not applicable		$88\epsilon^2$

Table 1: Width-to-thickness ratios for local buckling

Notes:

1. $\epsilon = \left(\frac{230}{f_y}\right)^{0.5}$

2. The stress ratios r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}}$$

$$r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress ofoverall section}}$$

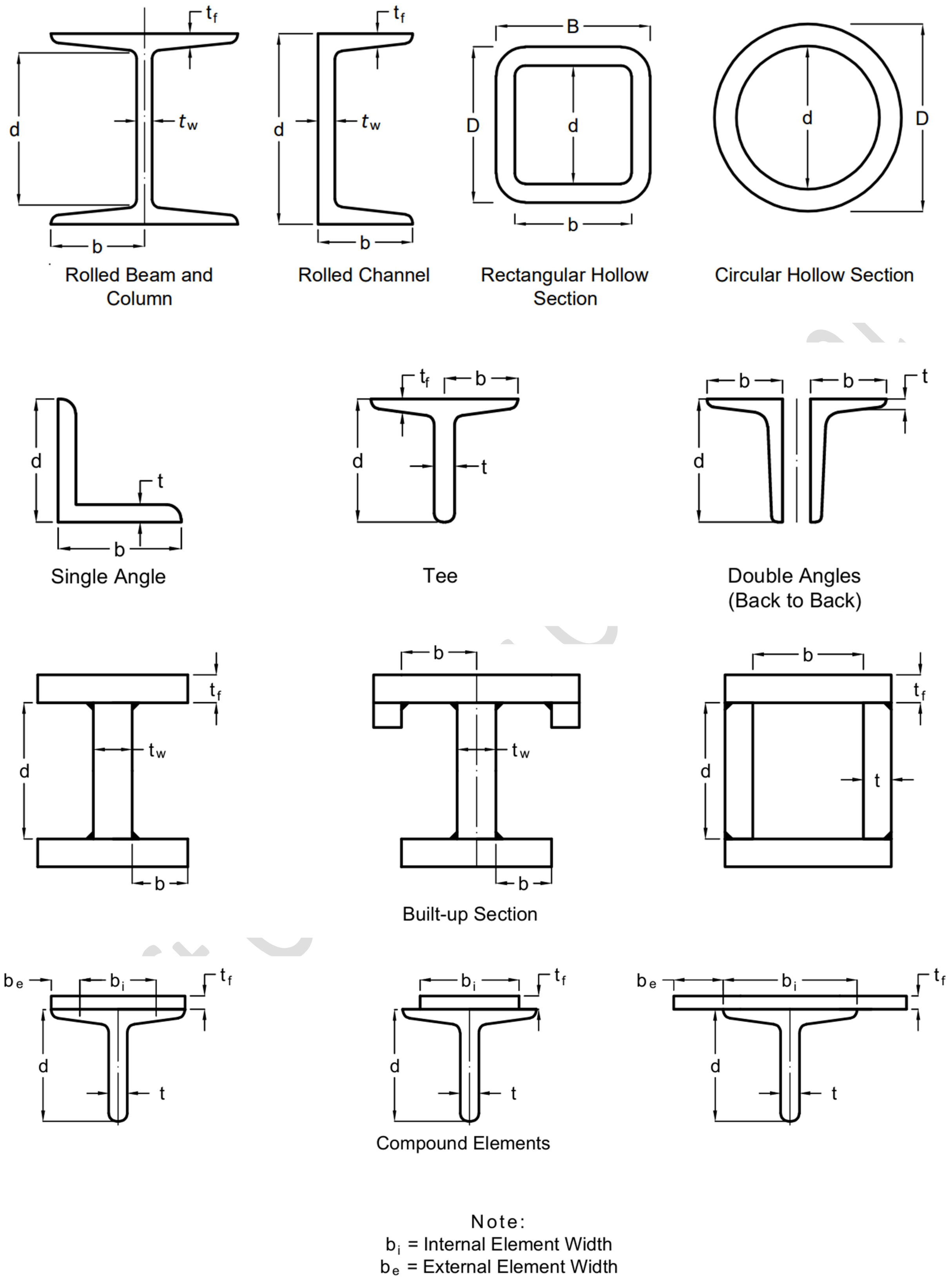


Figure 4: Dimensions of Section

3.6 Slenderness Ratio Limits

The maximum effective slenderness ratio, KL/r, values for different components are given in the Table 2. 'KL' is the effective length of the members and 'r' is appropriate radius of gyration based on the effective section.

Table 2: Slenderness Ratio Limits

SN	Member	Maximum Effective slenderness ratio (KL/r)
i)	Compressive member under dead and imposed loads	150
ii)	Tension member where load reversal occurs by forces other than wind or seismic load	150
iii)	Tension member subjected to load reversal due to wind or earthquake load	250
iv)	Compression flange of a beam subjected to lateral torsional buckling	300
v)	Member always under tension (other than pre-tensioned members)	300

3.7 Lateral Load Resistance

3.7.1 The steel structure designed shall possess adequate lateral load resistance capacities in case of lateral loads such as earthquake, wind, etc. The possibility of load reversal shall also be kept in due consideration.

3.7.2 In addition to adequate strength and rigidity under these forces, the steel framework shall also be designed such that vibration or sway is reduced to an acceptable standard. This can be achieved by using triangulated bracing or rigid porta systems.

3.8 Expansion Joints

3.8.1 Because of a large number of factors involved in issues of expansion and contraction, the task of deciding and locating an expansion joint is left to the discretion of the designing engineer.

3.8.2 Figure 5 is provided to serve as guide in deciding when an expansion joint is necessary to be provided.

3.8.3 Modification to the values obtained from the above plot is needed as follows:

i) If the structure is heated only and will have hinged column bases, use the allowable length as specified

ii) If the structure is air-conditioned as well as heated, increase the allowable length by 15% (provided the environmental control system will run continuously)

iii) If the building will be unheated, decrease the allowable length by 33%.

iv) If the building will have fixed column bases, decrease the allowable length by 13%.

v) If the building will have substantially greater stiffness against lateral displacement at one end of the plan dimension, decrease the allowable length by 25%.

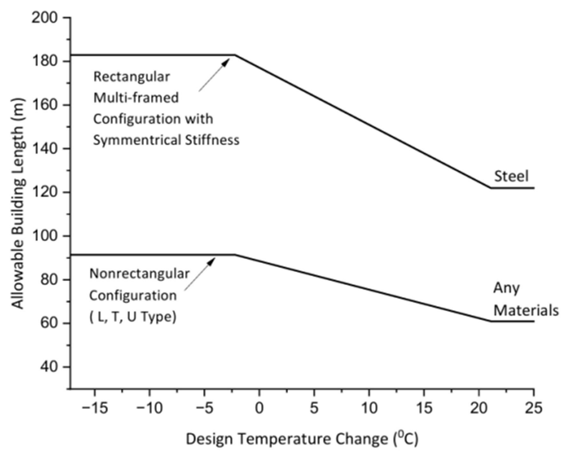


Figure 5: Maximum allowable building length without expansion joints for various design temperature changes

4 Methods of Structural Analysis

4.1 Methods of Determining Action Effects

4.1.1 General

For the purpose of complying with the requirements of the limit states of stability, strength and serviceability specified in section 5 effects of design actions on a structure and its members and connections, shall be determined by structural analysis using the assumptions of 4.2 and 4.3 and one of the following methods of analysis:

a) Elastic analysis in accordance with 4.4
b) Plastic analysis in accordance with 4.5,
c) Advanced analysis in accordance with Annex A, and
d) Dynamic analysis in accordance with NBC 105:2020.

4.1.2 Design Action Effects for Earthquake Loads

NBC 105 to be referred to for more information on design action effects for earthquake loads.

4.1.3 Braced Frame and Sway frames

For the purpose of analysis and design, the structural frames shall be classified as braced and sway frames.

4.1.3.1 **Braced frame–** A structure or structural frame shall be classified as “Braced” if its sway deformation is sufficiently small, such that the resulting secondary forces and moments are negligible.

This classification applies to:

a) triangulated frames and trusses
b) Frames where in-plane stiffness is provided by diagonal bracings, shear walls, floor slabs, or roof decks secured horizontally to walls or bracing systems parallel to the plane of buckling and bending of the frame.

A rigid jointed multi-story frame may be considered as a braced frame if in every individual story, the deflection, δ, over a story height, h_s, due to the notional horizontal loading given in 4.3.6 satisfies the following criteria:

a) For clad frames where the stiffening effect of the cladding is not taken into account in the deflection calculations: $\delta \leq \frac{h_s}{2000}$
b) For unclad frame or clad frames where the stiffening effect of the cladding is taken into account in the deflection calculations: $\delta \leq \frac{h_s}{4000}$
c) A frame, which when analyzed considering all the lateral supporting system does not comply with the above criteria, shall be classified as a sway frame, even if it is braced or otherwise laterally stiffened.

4.1.3.2 Sway frame **–** A structure or structural frame shall be classified as "Sway" if the transverse displacement of one end of a member relative to the other end is not effectively prevented, resulting in significant secondary forces and moments.

4.2 Forms of Construction Assumed for Structural Analysis

4.2.1 The effects of design actions on the members and connections of a structure shall be determined by assuming one or a combination of the following forms of construction.

4.2.1.1 Rigid Construction

For rigid construction, it shall be assumed that the connections maintain the original angles between the members effectively unchanged until the nominal capacity of the weakest member is reached.

4.2.1.2 Semi-rigid Construction

For semi-rigid construction, the connections shall not be required to possess sufficient rigidity to maintain the original angles between the members up to the attainment of the nominal capacity of the weakest member. However, the connections shall be required to provide a dependable and known degree of flexural restraint under the design actions. The relationship between the degree of flexural restraint and the level of member action shall be established by rational analysis (See Annex A), or experimentally.

4.2.1.3 Simple Construction

For simple construction, the connections between members shall be assumed to develop negligible bending moment between the connected members.

4.2.1.4 Design of Connections

The design of all connections shall be consistent with the form of construction, and the behavior of the connections shall not adversely affect any other part of the structure beyond what is allowed for in design. Connections shall be designed in accordance with Section 10,

4.2.2 Applicability of forms of construction for use in seismic resisting systems

4.2.2.1 In a braced seismic-resisting system any of the three forms of construction from 4.2.1 may be used.

4.2.2.2 In a moment-resisting framed seismic-resisting system, either rigid or semi-rigid construction shall be used.

4.3 Assumptions and Approximations for Analysis

4.3.1 The structure shall be analyzed in its entirety except as follows:

a) Regular building structures may be analyzed as a series of parallel two-dimensional frames, the analysis being carried out in each of the two directions at right angles, except where there is significant load redistribution between the frames. For seismic and wind loading, the design actions on each element of the lateral load-resisting system shall be determined from the response of the structure as a whole to the applied loads.

b) For vertical loading in a multi-story building structure, provided with bracing or shear walls to resist all lateral forces, each level thereof, together with the columns immediately above and below, may be considered as a sub structure, the columns being assumed fixed at the ends remote from the level under consideration.

Where beams at a floor level in a multi-bay building structure are considered as a sub-structure (part of a structure), the bending moment at the support of the beam due to gravity loads may be determined based on the assumption that the beam is fixed at the far end support, one span away from the span under consideration, provided that the floor beam is continuous beyond that support point.

4.3.2 Span Length

The span length of a flexural member in a continuous frame system shall be taken as the distance between center to center of the supports.

4.3.3 Arrangements of Live Loads in Buildings

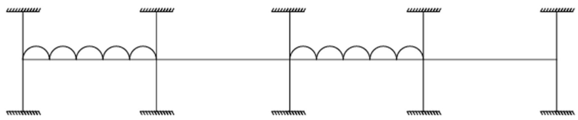
For building structures, the various arrangements of variable loads, considered for the analysis, shall include at least the following:

a) Where the loading pattern is fixed, the arrangement concerned.

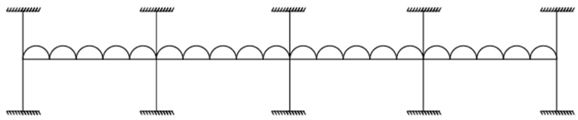
b) Where the live load is variable and not greater than three-quarters of the dead load, the live load may be taken to be acting on all spans.

c) Where the live load is variable and exceeds three-quarters of the dead load, arrangements of live load acting on the floor under consideration shall include the following cases:

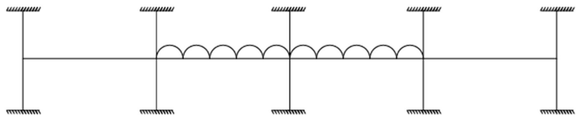
1) the live load on alternate spans;
2) the live load on two adjacent spans; and
3) the live load on all the spans.



Live load on Alternate Span



Live load on All Span



Live load on Adjacent Span

Figure 6: Cases of loading

4.3.4 Base Stiffness

In determining the stiffness of a base, consideration shall be given to the behavior of the ground, the stiffness of the foundation itself, and the characteristics of the steel baseplate or other connections. The stiffness of a base with a pin or a rocker shall be assumed to be zero.

In cases where detailed knowledge of the base stiffness is lacking, design assumptions may be made as follows:

a) When the column is rigidly connected to a suitable foundation, the stiffness of the pedestal shall be taken as the stiffness of the column above base plate. However, in case of very stiff pedestals and foundations column may be assumed as fixed at base.
b) When the column is nominally connected to the foundation, a pedestal stiffness of 10 percent of the column stiffness may be assumed.
c) When an actual pin or rocker is provided in the connection between the steel column and pedestal, the column is assumed as hinged at base and the pedestal and foundation may be appropriately designed for the reactions from the column.
d) In case of (a) and (b), the bottom of the pedestal shall be assumed to have the following boundary condition in the absence of any detailed procedure based on theory or tests:
   i) When the foundation consists of a group of piles with a pile cap, raft foundation or an isolated footing resting on rock or very hard soil, the pedestal shall be assumed to be fixed at the level of the bottom of footing or at the top of pile cap.
   ii) When the foundation consists of an isolated footing resting on other soils, pedestal shall be assumed to be hinged at the level of the bottom of footing.
   iii) When the pedestal is supported by a single pile, which is laterally surrounded by soil providing passive resistance, the pile shall be assumed to be fixed at a depth of 5 times the diameter of the pile below the ground level in case of compact ground or the top level of compact soil in case of poor soil overlying compact soil.
   iv) When the column is founded into rock, it may be assumed to be fixed at the interface of the column and rock.

4.3.5 Simple Construction

Bending members may be assumed to have their ends connected for shear only and to be free to rotate. In triangulated structures, axial forces may be determined by assuming that all members are pin connected. The eccentricity for stanchion and column shall be assumed in accordance with 7.3.3.

a) A beam reaction or a similar load on a column shall be taken as acting at a minimum distance of 100 mm from the face of the column towards the span or at the center of bearing, whichever gives the greater eccentricity, except that for a column cap, the load shall be taken as acting at the face of the column, or edge of packing, if used, towards the span.
b) In a continuous column, the design bending moment due to eccentricity of loading at any one floor or horizontal frame level shall be taken as:
   i) Ineffective at the floor or frame levels above and below that floor; and

ii) Divided between the columns above and below the floor or frame level in proportion to the values of I/L of the columns meeting at the junction.

4.3.6 Notional Horizontal loads

To analyze a frame subjected to gravity loads while considering its sway stability, notional horizontal forces shall be applied. These forces account for practical imperfections and shall be taken at each level as being equal to 0.5 percent of the factored dead load plus vertical imposed loads applied at that level. The notional loads shall not be applied along with other lateral loads such as wind and earthquake loads in the analysis.

4.4 Elastic Analysis

4.4.1 Assumptions

Individual members shall be assumed to remain elastic under the action of the factored design loads for all limit states. The effect of haunching or any variation of the cross-section along the axis of a member shall be considered, and where significant, shall be taken into account in the determination of the member stiffness.

4.4.2 First-order Elastic Analysis

In a first-order elastic analysis, the equilibrium of the frame in the un-deformed geometry is considered, the changes in the geometry of the frame due to the loading are not accounted for, and changes in the effective stiffnesses of the members due to axial force are neglected. The effects of these on the first-order bending moments shall be allowed for by using one of the methods of moment amplification of 4.4.3.2 or 4.4.3.3 as appropriate. Where the moment amplification factor $\mathrm{K_y}, \mathrm{K_z}$, calculated in accordance with 4.4.3.2 or 4.4.3.3 as appropriate, is greater than 1.4, a second-order elastic analysis in accordance with Annex A shall be carried out.

4.4.3 Second-order Elastic Analysis

4.4.3.1 The analysis shall allow for the effects of the design loads acting on the structure and its members in their displaced and deformed configuration. These second-order effects shall be taken into account by using either

a) A first-order elastic analysis with moment amplification in accordance with 4.4.3, provided the moment amplification factors $\mathrm{K_y}$ and $\mathrm{K_z}$ are not greater than 1.4; or

b) A second-order elastic analysis in accordance with Annex A.

4.4.3.2 Moment Amplification for Members in Braced Frames

For a member with zero axial compression or a member subject to axial tension, the design bending moment is that obtained from the first order analysis for factored loads, without any amplification.

For a braced member with a design axial compressive force P_d as determined by the first order analysis, the design bending moment shall be calculated considering moment amplification.

4.4.3.3 Moment Amplification for Member in a Sway Frames

The design bending moment shall be calculated as the product of moment amplification factor, (Section 9) and the moment obtained from the first order analysis of the sway frame, unless a more detailed analysis is carried out.

The calculated bending moments from the first order elastic analysis may be modified by redistribution up to 15% of the peak calculated moment of the member under factored load, provided that:

a) The internal forces and moments in the members of the frame are in equilibrium with applied loads.

b) All the members in which the moments are reduced belong to plastic or compact section classification. (Section 3.5.2).

4.5 Plastic Analysis

4.5.1 Application

The design action effects throughout or part of a structure may be determined by a plastic analysis, provided that the requirements of 4.5.2 are met. The distribution of design action effects shall satisfy equilibrium and the boundary conditions.

4.5.2 Requirements

When a plastic method of analysis is used, all of the following conditions of this section shall be satisfied, unless adequate ductility of the structure and plastic rotation capacity of its members and connections are established for the design loading conditions by other means of evaluation:

a) The yield stress for the grade of the steel used shall not exceed 450 MPa.
b) The stress-strain characteristics of the steel shall satisfy the following requirements as shown in Figure 7.
   i) the stress strain diagram has a plateau at the yield stress, extending for at least six times the yield strain;
   ii) the ratio of the tensile strength to the yield stress specified for the grade of the steel is not less than 1.2;
   iii) the elongation on a gauge length is not less than 15%; and
   iv) the steel exhibits strain-hardening capability.
c) The members used shall be hot-rolled or fabricated using hot-rolled plates and section.

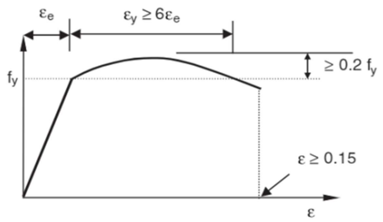


Figure 7: Stress strain diagram of steel exhibiting plastic characteristics

d) The cross section of members not containing plastic hinges shall be compact section, unless the members meet the strength requirements from elastic analysis.
e) Where plastic hinges occur in a member, the proportions of its cross section shall not exceed the limiting values for plastic section.
f) The cross section shall be symmetrical about its axis perpendicular to the axis of the plastic hinge rotation.
g) The members shall not be subject to impact loading, requiring fracture assessment or fluctuating loading, requiring a fatigue assessment.

4.5.2.1 Restraints

If practicable, torsional restraint (against lateral buckling) shall be provided at all plastic hinge locations. Where not feasible, the restraint shall be provided within a distance of D/2 of the plastic hinge location, where D is the total depth of section.

The torsional restraint requirement at a section as above need not be met at the last plastic hinge to form, provided it can be clearly identified.

Within a member containing a plastic hinge, the maximum distance L_m from the restraint at the plastic hinge to an adjacent restraint shall be calculated by any rational method or the conservative method given below, so as to prevent lateral buckling.

Conservatively L_m (in mm) may be taken as:

$$L_m \leq \frac{38\, r_y}{\left[\frac{f_c}{130} + \left(\frac{f_y}{250}\right)^2 \left(\frac{x_t}{40}\right)^2\right]^{1/2}}$$

where, f_c = average compressive stress on the cross section due to axial load (in N/mm^2)
f_y = yield stress (in N/mm^2); r_y = radius of gyration about the minor axis (in mm)
x_t = torsional index; A = area of cross section
I_W, I_y, I_t = warping constant, second moment of the cross section above the minor axes and St. Venant's torsion constant, respectively

Where the member has unequal flanges, r_y should be taken as the lesser of the values of the compression flange only or the whole section.

Where the cross section of the member varies within the length L_m, the maximum value of r_y and the maximum value of x_t shall be used.

The spacing of restraints to member lengths not containing a plastic hinge shall satisfy the recommendations of section on lateral buckling strength of beams (Section **8**). Where the restraints are placed at the limiting distance L_m, no further checks are required.

4.5.2.2 Stiffeners at Plastic Hinge Locations

Web stiffeners shall be provided where a concentrated load is applied within D/2 of a plastic hinge location, which exceeds 10% of the shear capacity of the member (see 8.2.1.2). The stiffener shall be provided within a distance of half the depth of the member, on either side of the hinge location and be designed to carry the applied load. If the stiffeners are flat plates, the outstand width to the thickness ratio, b/t, should not exceed the values given in the plastic section (3.5.2). Where such sections are used the ratio $\left(\frac{I_{SO}}{I_t}\right)^{\frac{1}{2}}$, should not exceed the values given for plastic section (for simple outstand in Section 3.5.2).

where, $I_{SO} =$ second moment of area of the stiffener about the face of the element perpendicular to the web,

$I_t =$ St. Venant's torsion constant of the stiffener.

4.5.2.3 Fabrication Restriction

Within a length equal to the member depth, on either side of a plastic hinge location, the following restrictions shall be applied to the tension flange and noted in the design drawings.

a) Holes if required, shall be drilled or else punched 2 mm undersize and reamed
b) All sheared or hand flame cut edges shall be finished smooth by grinding, chipping or planning.

4.5.3 Assumptions in Analysis

The design action effects shall be determined using a rigid- plastic analysis.

It shall be permissible to assume full strength or partial strength connections, provided the capacities of these are used in the analysis, and provided that:

a) In a full-strength connection, the moment capacity of the connection shall be not less than that of the member being connected,
b) In a partial strength connection, for which the moment capacity of the connection may be less than that of the member being connected,
c) In both cases the behavior of the connection shall be such as to allow all plastic hinges necessary for the collapse mechanism to develop, and shall be such that the required plastic hinge rotation does not exceed the rotation capacity at any of the plastic hinges in the collapse mechanism.

In the case of building structures, it is not normally necessary to consider the effect of alternating plasticity.

4.5.4 Second-order Elastic Analysis

Any second-order effects of the loads acting on the structure in its deformed configuration may be neglected, provided the following are satisfied:

a) For clad frames, provided the stiffening effects of masonry infill wall panels or diaphragms of profiled wall panel is not taken into account, and where elastic buckling load factor, λ_{cr}, satisfies $\frac{\lambda_{cr}}{\lambda_p} \geq 10$. If $10 > \frac{\lambda_{cr}}{\lambda_p} \geq 4.6$, the second-order effects may be considered by amplifying the design load effects obtained from plastic analysis by a factor $\delta_p = \{0.9\,\lambda_{cr}/(\lambda_{cr} - 1)$. If $\frac{\lambda_{cr}}{\lambda_p} < 4.6$, second-order elasto-plastic analysis or second-order elastic analysis is to be carried out.

b) For un-clad frames or for clad frames where the stiffening effects of masonry infill or diaphragms of profiled wall panel is taken into account, where elastic buckling load factor λ_{cr}, satisfies $\frac{\lambda_{cr}}{\lambda_p} \geq 20$. If $20 > \frac{\lambda_{cr}}{\lambda_p} \geq 5.75$, the second-order effects may be considered by amplifying the design load effects obtained from plastic analysis by a factor $\delta_p = \frac{0.9\,\lambda_{cr}}{\lambda_{cr}-1}$. If $\frac{\lambda_{cr}}{\lambda_p} < 5.75$, second order elasto-plastic analysis or second-order elastic analysis shall be carried out.

4.6 Frame Buckling Analysis

4.6.1 The elastic buckling load factor (λ_{cr}) shall be the ratio of the elastic buckling load set of the frame to the design load set for the frame, and shall be determined in accordance with 4.6.2

Note: The value of (λ_{cr}) depends on the load set and has to be evaluated for all the possible sets of load combinations.

4.6.2 In-plane frame buckling

The elastic buckling load factor (λ_{cr}) of a rigid-jointed frame shall be determined by using:

a) One of the approximate methods of 4.6.2.1 and 4.6.2.2; or
b) A rational elastic buckling analysis of the whole frame.

4.6.2.1 Rectangular frames with all members braced

In a rectangular frame with regular loading and negligible axial forces in the beams, the Euler buckling stress $f_{cc,}$ for each column shall be determined in accordance with 7.1.2.1 .The elastic buckling load factor (λ_{cr}) for each column shall be taken as the lowest of the ratio of (f_{cc}/f_{cd}) for all the columns.Wheref_{cc} is the elastic buckling stress of the column and f_{cd} is the axial compression stress in the column from the factored load analysis.

4.6.2.2 Rectangular frames with sway members

In a rectangular frame with regular loading and negligible axial forces in the beams, the buckling load, P_{cc}, for each column shall be determined as P_{cc} = A f_{cc}, where f_{cc}, is the elastic buckling stress of the column in the plane of frame, obtained in accordance with 7.1.2.1. The elastic buckling load factor λ_{cr}, for the whole frame shall be taken as the lowest of all the ratios, λ_{scr}, calculated for each story of the building, as given below:

$$\lambda_{scr} = \frac{\sum\left(\frac{P_{cc}}{L}\right)}{\sum\left(\frac{P}{L}\right)}$$

where, P = member axial force from the factored load analysis, with tension taken as negative

L = column length and the summation include all columns within a story.

4.7 Performance-Based Design

4.7.1 General

Performance-Based Design (PBD) is an advanced engineering approach that focuses on designing structures to achieve specific performance objective under extreme events such as earthquake. This section outlines the principles and procedures for implementing PBD in the design of steel structures, ensuring that they meet the required performance standards under different levels of seismic demand.

4.7.2 Objectives of performance-based design

The primary objectives of PBD are:

a) To ensure the safety and functionality of steel structures under different seismic events
b) To provide a framework for designing buildings that can achieve predictable performance levels
c) To offer flexibility in design solution allowing for innovative approaches that meet performance criteria.

4.7.3 Performance levels

Performance levels define the expected condition of a building after a seismic event. The common performance levels are:

a) Immediate Occupancy (IO): Minimal damage, fully operational
b) Life Safety (LS): Significant damage, but no collapse; life safety is protected
c) Collapse prevention (CP): Near Collapse but no catastrophic failure

4.7.4 Criteria for conducting performance-based design

When the structure is deemed to be complex, novel or to hold significant uncertainty in its performance, performance-based design (via non-linear modelling and dynamic analyses- refer NBC 105) must be conducted and reviewed through expert consultations.

5 Limit State Design

5.1 Philosophy

Within the limit state design framework, a structure is designed for achieving certain limit states of strength and serviceability throughout its lifetime. These limit states are acceptable limits of how much strength a structure shall possess and what acceptable serviceability shall the structure possess.

A designed steel structure shall behave as a one three-dimensional entity. The connections and elements used shall constitute a structure that must be stable under normal loading and under accidental loading shall not suffer progressive collapse.

5.2 Limit State Design

The basic idea behind Limit State Design is to satisfy the following equation:

Design Action $\leq$ Design Strength $\sum_i \gamma_i P_i \leq \phi R_n$

where, P_i=nominal load on the structure, γ_i= load factors, R_n=nominal member capacity, ϕ = capacity factors for structure resistance.

Thus, by accounting for variations of loadings on structure and variation of resistance offered by structure, the limit state design achieves reliability in design.

The limit states considered by this code are classified as:

a) Limit state of strength

The following instances specify the limit state of strength:

i) Loss of equilibrium of a part or whole of the structure.
ii) Loss of load bearing capacity of structure evident by excessive deformation, rupture of the structure or any part.
iii) Fracture due to fatigue
iv) Brittle facture

b) Limit State of Serviceability

The following instances specify the limit state of serviceability:

i) Excessive deformation of the structure even when the load is being beard
ii) Excessive vibrations in the structure
iii) Corrosion, durability
iv) Damage due to Fire
v) Excessive cracking

5.3 Loads

All kinds of loading sustained on a structure during its lifetime shall be considered. Primary loads comprise of dead loads of structural components and non-structural non-components making up the steel structure. Secondary loads comprise of loads during construction, live loads, wind loads, construction loads, etc.

5.3.1 Characteristics Load

Characteristic load is that load which has a 95 % probability of not being exceeded during its lifetime.

This code recognizes the following loads as characteristics load for the design of steel structures:

a) Dead load of the structure adhering to NBC 102.
b) Live load on the structure adhering to NBC 103.
c) Additional loads that may be expected on the structure as specified by the client and in fulfillment of minimum provisions of respective load standard

Design loads shall be increased as per load types and combinations and limit states using factors as specified in Table 3.

Table 3: Load factors, γ_i for various load types and combinations

Combinations	Limit State of Strength			Limit State of Serviceability		
	DL	LL	WL/EL	DL	LL	WL/EL
DL + LL + CL	1.2	1.5	-	1	1	-
DL + LL + CL + WL/EL	1.2	0.6[1]	1	1	0.8	0.8
DL + WL/EL	1.2 (0.9)[2]		1	-	-	-
DL + ER	1.2 (0.9)[2]	1.5	-	-	-	-

Note: 1: use 0.6 when live load is of storage type and 0.3 when live load is of non-storage type
2: whenever dead load acts to reduce stresses caused due to other loading

5.4 Strength

The design strength S_d of a structure is obtained by the use of partial safety factor for material strength, γ_m as given in Table 4.

$$S_d = S_u/\gamma_m$$

Table 4: Partial Safety Factors for Material, ϕ_m

S N	Definitions	Partial Safety Factors	
1	Partial safety factor for yielding, γ_{m0}	1.1	
2	Partial safety factor for buckling, γ_{m0}	1.1	
3	Partial safety factor for ultimate stress, γ_{m1}	1.25	
4	Resistance of Connections	**Shop Fabrications**	**Site Fabrications**
	a) Friction type bolts, γ_{mf}	1.25	1.25
	a) Bearing type bolts, γ_{mb}	1.25	1.25
	b) Welds, γ_{mw}	1.25	1.5

5.5 Limit States of Strength

Following limit states shall be considered while evaluating a steel structures limit state of strength:

a) Static equilibrium: The static equilibrium of a structure means that the frame is stable against sliding and overturning and uplift. The stabilizing action against each of these scenarios shall be greater than the loads causing them in order to satisfy static equilibrium.

b) Sway Stability: The structure shall be adequately stiff against sway which might induce structural damage or even encroach limits of serviceability.

c) Fatigue: When a structure is repeatedly subjected to reversal of stresses throughout its lifetime, fatigue needs to be considered as a limit state of strength.

d) Ductile failure mode: When local buckling is prevented and members are also restrained laterally in case of members experiencing flexure, ductile failure mode can be considered through the formation of plastic hinges.

5.6 Limit States of Serviceability

In addition to strength, a structure is only of use when it is serviceable. The following factors are checked to ensure serviceability in a steel structure:

a) Deflection: The deflection values which mark the limit of serviceability for various steel structures and components are listed in Table 5.

b) Vibration: For structures with possibility of vibrations checks shall be done to ensure that resonance of the structure is not a possibility. Wind induced vibrations shall be checked in flexible structures (when height is 4 times the lateral width in a lateral force resisting system). Floor vibration effects shall be considered using specialist literature.

c) Durability: A steel structure must stand the adverse conditions of the environment in which it is built. Environment surrounding a steel structure, exposure, protective maintenance, etc. come into play when considering durability. Minimum guidelines for increasing durability in steel structures are given in Section 14. Specialist literatures shall be referred in order to increase durability requirements.

d) Fire: A structure's resistance to fire is dependent on a wide variety of factors including but not limited to its mass, geometry and support condition. Minimum provision for fire resistance of steel structure components is given in Section (needs separate heading and is yet to be written).

Specialist literatures shall be referred in order to increase fire resistance.

Table 5: Deflection Limits

Type of Building	Condition	Loads	Component	Supporting	Maximum Deflection
Industrial Buildings	Vertical	Live load/Wind Load	Purlin and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple Span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever Span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/wind load	Rafter supporting	Profiled metal sheeting	Span/180
				Plastered sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	40mm
		Crane + wind	Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Building	Vertical	Live load	Floor and Roof	Elements not susceptible to cracking	Span/360
				Elements susceptible to cracking	Span/300
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180
	Lateral	Wind	Building	Elastic cladding	Height/300
				Brittle cladding	Height/500
		Wind	Inter Story drift	-	Story height/300

Note: live loads shall include all post construction load including super imposed dead loads

6 Design of Member for Pure Axial Tension

6.1 General

When a structural member is subjected to axial tension along its longitudinal axis without any eccentricity, the case is known as pure axial tension.

The design of a member experiencing axial tension is governed by three limit states:

a) Gross Section Yielding
b) Net Section Rupture
c) Block Shear Fracture

The design tension strength of a member shall exceed the greater of the limiting strength required considering the above three limit states. When it is possible, it is recommended to make gross section yielding as the most critical limit state for tension member design.

6.2 Gross Section Yielding

The design strength of members under axial tension, governed by gross section yielding is given by:

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where, A_g= gross area of cross-section,
f_y = yield strength of material under tension
γ_{m0}= partial safety factor for yielding (See Table 4).

6.3 Net Section Rupture

6.3.1 Plates

The design strength in net section rupture of a plate, T_{dn} is given by as governed by rupture of net cross-sectional area, A_n, at the holes is given by

$$T_{dn} = 0.85\, A_n f_u / \gamma_{m1}$$

where, γ_{m1}= Partial safety factor for failure at ultimate state stress

f_u = Ultimate stress of material

A_n= net effective area of members, given by

$$A_n = \left[b - n d_n + \sum_i \frac{p_{si}^2}{4 g_i} \right] t$$

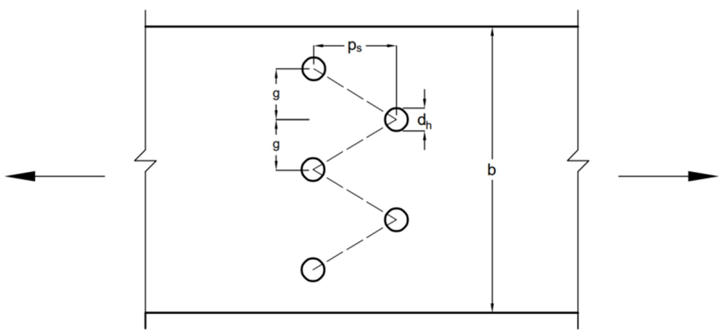


Figure 8: Plate under tension

where, b , t = width and thickness of plate respectively,

d_n = diameter of bolt hole (2 mm in addition to the diameter of the hole in case of directly punched holes),

g = gauge length between the bolt holes,

p_s = staggered pitch length between lines of bolt holes,

n = number of bolt holes in critical section,

i = subscript for summation of all the inclined legs,

6.3.2 Threaded Rods

The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by:

$$T_{dn} = 0.85\, A_n f_u / \gamma_{m1}$$

where, A_n = net root area at the threaded section.

6.3.3 Single angles, Double angles, I-section and channels

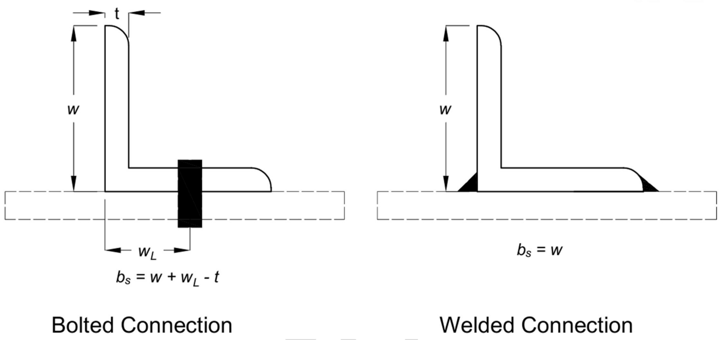


Figure 9: Angle connections

The rupture strength of a single angles, double angle, I-section and channels is affected by shear lag. The design strength, T_{dn} as governed by rupture at net section is given by:

$$T_{dn} = \frac{0.85 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

where, $\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \quad \le \left(0.9 \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right)$

≥ 0.7

w = outstand leg width; b_s = shear lag width

L_c = length of the end connection

For preliminary sizing, the rupture strength of net section may be approximately taken as:

$$T_{dn} = \alpha A_n f_u / \gamma_{m1}$$

where, α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length;

A_n = net area of the total cross-section; A_{nc}= net area of connected leg;

A_{go}= gross area of the outstanding leg; t=thickness of leg.

6.4 Design Strength Due to Block Shear

The strength as governed by block shear at an end connection of plates and angles is calculated as given by 6.4.1 and 6.4.2.

6.4.1 Bolted Connections

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.85 A_{tn} f_u}{\gamma_{m1}} \quad \text{or} \quad T_{db} = \frac{0.85 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

where, A_{vg}= minimum gross area in shear along bolt line parallel to external force;
A_{vn}= minimum net area in shear along bolt line parallel to external force;
A_{tg}= minimum gross area in tension from the bolt hole to the toe of the angle in bolt line, perpendicular to the line of force;
A_{tn}= minimum net area in tension from the bolt hole to the toe of the angle in bolt line, perpendicular to the line of force;
f_u= ultimate stress of the material; f_y= yield stress of the material

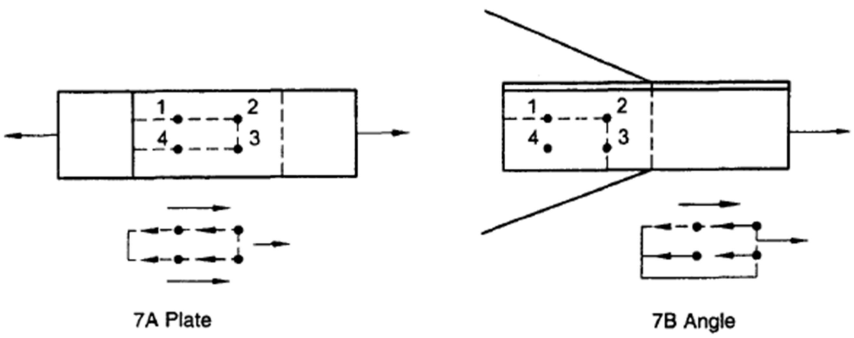


Figure 10: Block Shear Failure

6.4.2 Welded Connections

The block shear strength, T_{db} shall be checked for welded end connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

6.5 Laced or Battened Ties

When it is required such that two members shall be used using lacing or battens to create built up tension members, the following provisions shall be followed:

a) The lacing or battens shall be designed to resist the greater of:
   i) Axial forces, moments and shear forces induced by eccentric loads, applied moments or transverse forces, including self-weight and wind resistance.
   ii) Axial forces, moments and shear forces induced by a transverse shear on the complete member at any point in its length equal to 1% of the axial force in the member, taken as shared equally between all transverse lacing or battening system in parallel planes.
b) For the design of lacing or batten, compliance must be ensured to (general compression member guidelines) except as follows:
   i) The slenderness ratio of lacing element shall not exceed 210.
   ii) The slenderness of unconnected part of the member shall not exceed 300.
   iii) Batten plates shall have a thickness of not less than 0.017 times the distance between the innermost lines of connections.
   iv) Intermediate battens shall have a width of not less than half the effective width of end batten plates.

7 Design of Member for Compression

7.1 General

7.1.1 Design of compression members is mainly influenced by a phenomenon named as buckling (local buckling and member buckling). Assuming local buckling to be prevented by adopting provisions as per (earlier local buckling provisions), buckling determines the strength of compressive members. There can be different modes of buckling; Flexural buckling, torsional buckling and flexural torsional buckling.

The design compressive strength of a member except angle and double angle struts are assumed to limited by flexural buckling.

7.1.2 In general, the design compressive strength of a member P_d, is given by:

$$P < P_d$$

where, $P_d = A_e f_{cd}$

$A_e =$ effective sectional area as per

$f_{cd} =$ design compresive stress

7.1.2.1 The design compressive stress, f_{cd} of an axially loaded compression member considering flexural buckling limit state is calculating using:

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \gamma^2]^{0.5}} = \chi \frac{f_y}{\gamma_{m0}} \leq \frac{f_y}{\gamma_{m0}}$$

where, $\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$

$\lambda =$ non-dimensional effective slenderness ratio $= \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y}{\pi^2 E}\left(\frac{KL}{r}\right)^2}$

$f_{cc} =$ Euler buckling stress$= \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

where, $\frac{KL}{r} =$ effective slenderness ratio or ratio of effective length, KL to radius of gyration

$\alpha =$ imperfection factor as per

$\chi =$ stress reduction factor for different buckling class, slenderness ratio and yield stress

$$= \frac{1}{[\phi + (\phi^2 - \gamma^2)^{0.5}]}$$

$\gamma_{m0} =$ partial safety factor for material strength

7.1.2.2 The classification of different sections under different buckling classes a, b, c and d are given in the Table 6. The stress reduction factor χ, and the design compressive stress f_{cd}, for different bucking class, yield stress, and effective slenderness ratio is given in tables Table 7, Table 8, Table 9 and Table 10. The curves corresponding to different buckling class are presented in non-dimensional form.

Table 6: Imperfection factor, α

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

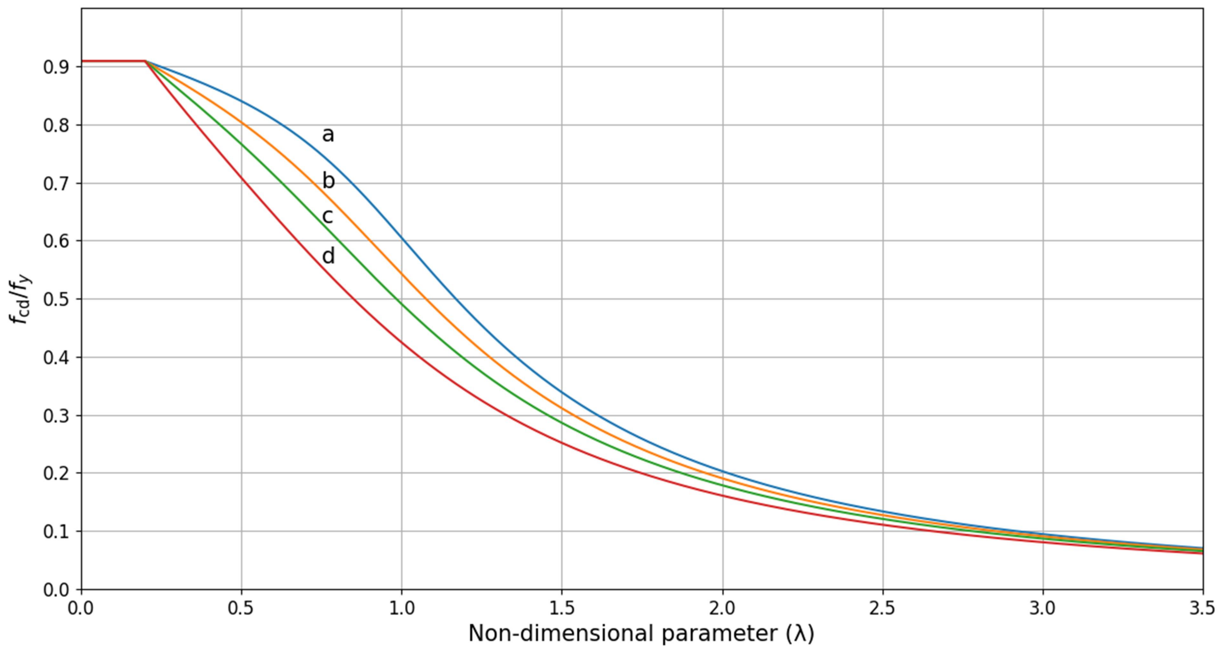


Figure 11: Buckling curves for column design

Table 7: Stress Reduction Factor, χ for Column Buckling Class a

KL/r	Yield Stress, f_y (MPa)																		
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**	**420**	**450**	**480**	**510**	**540**
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	1.000	0.999	0.998	0.997	0.995	0.994	0.993	0.992	0.990	0.988	0.986	0.984	0.983	0.981	0.979	0.977	0.975	0.972	0.970
30	0.977	0.975	0.974	0.972	0.970	0.969	0.967	0.964	0.961	0.957	0.954	0.951	0.948	0.946	0.943	0.938	0.934	0.930	0.925
40	0.952	0.949	0.947	0.944	0.942	0.939	0.937	0.931	0.926	0.921	0.916	0.911	0.906	0.901	0.896	0.888	0.881	0.873	0.865
50	0.923	0.919	0.915	0.911	0.908	0.904	0.900	0.892	0.884	0.876	0.867	0.859	0.851	0.842	0.834	0.820	0.807	0.794	0.780
60	0.888	0.883	0.877	0.871	0.865	0.859	0.853	0.841	0.828	0.816	0.803	0.790	0.777	0.763	0.750	0.730	0.710	0.690	0.671
70	0.846	0.837	0.829	0.820	0.811	0.803	0.794	0.776	0.758	0.740	0.722	0.703	0.686	0.668	0.651	0.626	0.602	0.579	0.557
80	0.793	0.781	0.769	0.757	0.746	0.734	0.722	0.698	0.675	0.653	0.631	0.610	0.589	0.570	0.551	0.525	0.501	0.478	0.458
90	0.730	0.715	0.700	0.685	0.671	0.657	0.643	0.615	0.590	0.565	0.542	0.520	0.500	0.481	0.463	0.439	0.416	0.396	0.377
100	0.661	0.644	0.627	0.610	0.594	0.579	0.564	0.536	0.510	0.486	0.463	0.443	0.424	0.407	0.390	0.368	0.348	0.331	0.314
110	0.591	0.573	0.555	0.538	0.522	0.507	0.492	0.465	0.440	0.418	0.397	0.379	0.362	0.346	0.332	0.312	0.295	0.279	0.265
120	0.525	0.507	0.489	0.473	0.458	0.443	0.429	0.404	0.381	0.361	0.343	0.326	0.311	0.297	0.284	0.267	0.252	0.238	0.226
130	0.466	0.448	0.432	0.416	0.402	0.388	0.376	0.353	0.332	0.314	0.298	0.283	0.269	0.257	0.246	0.231	0.217	0.206	0.195
140	0.413	0.397	0.382	0.368	0.355	0.342	0.331	0.310	0.291	0.275	0.260	0.247	0.235	0.224	0.214	0.201	0.189	0.179	0.170
150	0.368	0.353	0.339	0.326	0.314	0.303	0.293	0.274	0.257	0.243	0.229	0.218	0.207	0.197	0.189	0.177	0.166	0.157	0.149
160	0.329	0.316	0.303	0.291	0.280	0.270	0.261	0.244	0.229	0.215	0.204	0.193	0.184	0.175	0.167	0.157	0.147	0.139	0.132
170	0.296	0.283	0.272	0.261	0.251	0.242	0.233	0.218	0.204	0.192	0.182	0.172	0.164	0.156	0.149	0.140	0.131	0.124	0.117
180	0.267	0.255	0.245	0.235	0.226	0.218	0.210	0.196	0.184	0.173	0.163	0.155	0.147	0.140	0.134	0.125	0.118	0.111	0.105
190	0.242	0.231	0.222	0.213	0.205	0.197	0.190	0.177	0.166	0.156	0.147	0.140	0.133	0.126	0.121	0.113	0.106	0.100	0.095
200	0.220	0.210	0.202	0.193	0.186	0.179	0.172	0.161	0.151	0.142	0.134	0.127	0.120	0.115	0.109	0.102	0.096	0.091	0.086
210	0.201	0.192	0.184	0.177	0.170	0.163	0.157	0.147	0.137	0.129	0.122	0.115	0.110	0.104	0.099	0.093	0.087	0.083	0.078
220	0.184	0.176	0.169	0.162	0.155	0.149	0.144	0.134	0.126	0.118	0.111	0.106	0.100	0.095	0.091	0.085	0.080	0.075	0.071
230	0.170	0.162	0.155	0.149	0.143	0.137	0.132	0.123	0.115	0.108	0.102	0.097	0.092	0.088	0.083	0.078	0.073	0.069	0.065
240	0.157	0.149	0.143	0.137	0.132	0.127	0.122	0.114	0.106	0.100	0.094	0.089	0.085	0.081	0.077	0.072	0.068	0.064	0.060
250	0.145	0.138	0.132	0.127	0.122	0.117	0.113	0.105	0.098	0.092	0.087	0.082	0.078	0.074	0.071	0.066	0.062	0.059	0.056

Table 8: Stress Reduction Factor, χ for Column Buckling Class b

KL/r	**Yield Stress, f_y (MPa)**																		
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**	**420**	**450**	**480**	**510**	**540**
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	1.000	0.998	0.996	0.994	0.993	0.991	0.990	0.986	0.983	0.981	0.978	0.975	0.972	0.970	0.967	0.963	0.960	0.956	0.953
30	0.963	0.961	0.958	0.955	0.953	0.950	0.948	0.943	0.938	0.933	0.929	0.924	0.920	0.915	0.911	0.904	0.898	0.892	0.886
40	0.925	0.921	0.917	0.913	0.909	0.906	0.902	0.895	0.887	0.880	0.873	0.866	0.859	0.852	0.845	0.835	0.825	0.815	0.805
50	0.883	0.877	0.872	0.866	0.861	0.855	0.850	0.839	0.829	0.818	0.808	0.798	0.787	0.777	0.767	0.752	0.737	0.722	0.708
60	0.835	0.827	0.820	0.812	0.805	0.798	0.790	0.775	0.761	0.746	0.732	0.718	0.704	0.691	0.677	0.657	0.638	0.620	0.602
70	0.781	0.771	0.761	0.751	0.742	0.732	0.722	0.703	0.685	0.667	0.649	0.632	0.615	0.599	0.584	0.561	0.540	0.520	0.502
80	0.721	0.709	0.697	0.685	0.673	0.661	0.650	0.627	0.606	0.585	0.566	0.547	0.529	0.512	0.496	0.474	0.453	0.434	0.416
90	0.657	0.643	0.629	0.615	0.602	0.589	0.576	0.552	0.530	0.508	0.488	0.470	0.452	0.436	0.421	0.400	0.380	0.363	0.346
100	0.593	0.577	0.562	0.548	0.534	0.520	0.507	0.483	0.461	0.440	0.421	0.403	0.387	0.372	0.358	0.339	0.321	0.306	0.291
110	0.531	0.515	0.500	0.485	0.471	0.458	0.445	0.422	0.401	0.381	0.364	0.348	0.333	0.319	0.306	0.289	0.274	0.260	0.247
120	0.474	0.458	0.443	0.429	0.416	0.403	0.391	0.370	0.350	0.332	0.316	0.301	0.288	0.276	0.265	0.249	0.236	0.223	0.212
130	0.423	0.408	0.394	0.380	0.368	0.356	0.345	0.325	0.307	0.291	0.276	0.263	0.251	0.240	0.230	0.217	0.204	0.194	0.184
140	0.378	0.364	0.350	0.338	0.327	0.316	0.306	0.287	0.271	0.256	0.243	0.231	0.221	0.211	0.202	0.190	0.179	0.169	0.161
150	0.339	0.325	0.313	0.302	0.291	0.281	0.272	0.255	0.241	0.227	0.215	0.205	0.195	0.186	0.178	0.167	0.158	0.149	0.142
160	0.305	0.292	0.281	0.271	0.261	0.252	0.243	0.228	0.215	0.203	0.192	0.182	0.174	0.166	0.158	0.149	0.140	0.133	0.126
170	0.275	0.264	0.253	0.244	0.235	0.227	0.219	0.205	0.193	0.182	0.172	0.163	0.155	0.148	0.142	0.133	0.125	0.118	0.112
180	0.249	0.239	0.229	0.220	0.212	0.205	0.198	0.185	0.174	0.164	0.155	0.147	0.140	0.133	0.128	0.120	0.113	0.106	0.101
190	0.227	0.217	0.208	0.200	0.193	0.186	0.179	0.168	0.157	0.148	0.140	0.133	0.127	0.121	0.115	0.108	0.102	0.096	0.091
200	0.207	0.198	0.190	0.183	0.176	0.169	0.163	0.153	0.143	0.135	0.128	0.121	0.115	0.110	0.105	0.098	0.092	0.087	0.083
210	0.190	0.182	0.174	0.167	0.161	0.155	0.149	0.140	0.131	0.123	0.117	0.110	0.105	0.100	0.096	0.090	0.084	0.080	0.075
220	0.174	0.167	0.160	0.154	0.148	0.142	0.137	0.128	0.120	0.113	0.107	0.101	0.096	0.092	0.088	0.082	0.077	0.073	0.069
230	0.161	0.154	0.147	0.141	0.136	0.131	0.126	0.118	0.111	0.104	0.098	0.093	0.088	0.084	0.080	0.075	0.071	0.067	0.063
240	0.149	0.142	0.136	0.131	0.126	0.121	0.117	0.109	0.102	0.096	0.091	0.086	0.082	0.078	0.074	0.070	0.065	0.062	0.058
250	0.138	0.132	0.126	0.121	0.117	0.112	0.108	0.101	0.095	0.089	0.084	0.080	0.076	0.072	0.069	0.064	0.060	0.057	0.054

1922 *Table 9: Stress Reduction Factor, χ for Column Buckling Class c*

KL/r	Yield Stress, f_y (MPa)													
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	0.999	0.997	0.994	0.992	0.990	0.987	0.985	0.981	0.976	0.972	0.968	0.964	0.961	0.957
30	0.948	0.944	0.941	0.937	0.933	0.930	0.926	0.920	0.913	0.907	0.901	0.895	0.889	0.883
40	0.896	0.891	0.885	0.880	0.875	0.870	0.866	0.856	0.847	0.838	0.829	0.820	0.812	0.803
50	0.841	0.834	0.827	0.821	0.814	0.807	0.801	0.788	0.776	0.763	0.752	0.740	0.729	0.717
60	0.783	0.774	0.765	0.757	0.748	0.740	0.732	0.716	0.700	0.685	0.670	0.656	0.642	0.628
70	0.722	0.711	0.700	0.690	0.680	0.670	0.660	0.641	0.623	0.605	0.588	0.572	0.557	0.542
80	0.659	0.646	0.634	0.622	0.611	0.600	0.589	0.568	0.548	0.529	0.512	0.495	0.479	0.464
90	0.596	0.583	0.569	0.557	0.544	0.533	0.521	0.499	0.479	0.460	0.443	0.426	0.411	0.397
100	0.536	0.522	0.508	0.495	0.483	0.471	0.459	0.438	0.418	0.400	0.384	0.368	0.354	0.341
110	0.480	0.466	0.453	0.440	0.428	0.416	0.405	0.385	0.366	0.349	0.333	0.319	0.306	0.294
120	0.430	0.416	0.403	0.391	0.379	0.368	0.358	0.339	0.321	0.306	0.291	0.278	0.267	0.256
130	0.385	0.372	0.360	0.348	0.337	0.327	0.317	0.299	0.283	0.269	0.256	0.244	0.234	0.224
140	0.346	0.333	0.322	0.311	0.301	0.291	0.282	0.266	0.251	0.238	0.227	0.216	0.206	0.197
150	0.311	0.300	0.289	0.279	0.269	0.261	0.252	0.237	0.224	0.212	0.202	0.192	0.183	0.175
160	0.281	0.270	0.260	0.251	0.242	0.234	0.227	0.213	0.201	0.190	0.180	0.172	0.164	0.156
170	0.255	0.245	0.236	0.227	0.219	0.212	0.205	0.192	0.181	0.171	0.162	0.154	0.147	0.140
180	0.232	0.223	0.214	0.206	0.199	0.192	0.186	0.174	0.164	0.155	0.147	0.139	0.133	0.127
190	0.212	0.203	0.195	0.188	0.181	0.175	0.169	0.158	0.149	0.140	0.133	0.126	0.120	0.115
200	0.194	0.186	0.179	0.172	0.166	0.160	0.154	0.144	0.136	0.128	0.121	0.115	0.110	0.105
210	0.178	0.171	0.164	0.158	0.152	0.146	0.141	0.132	0.124	0.117	0.111	0.105	0.100	0.096
220	0.164	0.157	0.151	0.145	0.140	0.135	0.130	0.122	0.114	0.108	0.102	0.097	0.092	0.088
230	0.152	0.145	0.140	0.134	0.129	0.124	0.120	0.112	0.105	0.099	0.094	0.089	0.085	0.081
240	0.141	0.135	0.129	0.124	0.120	0.115	0.111	0.104	0.098	0.092	0.087	0.082	0.078	0.075
250	0.131	0.125	0.120	0.115	0.111	0.107	0.103	0.096	0.090	0.085	0.081	0.076	0.073	0.069

Table 10: Stress Reduction Factor, χ for Column Buckling Class d

KL/r	Yield Stress, f_y (MPa)																		
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**	**420**	**450**	**480**	**510**	**540**
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	0.999	0.995	0.991	0.988	0.984	0.980	0.977	0.970	0.964	0.958	0.952	0.946	0.940	0.935	0.930	0.922	0.915	0.908	0.901
30	0.922	0.916	0.911	0.906	0.901	0.896	0.891	0.881	0.872	0.863	0.855	0.847	0.839	0.831	0.823	0.813	0.802	0.792	0.782
40	0.848	0.841	0.834	0.828	0.821	0.815	0.808	0.796	0.784	0.773	0.762	0.751	0.741	0.731	0.721	0.707	0.694	0.681	0.668
50	0.777	0.768	0.760	0.752	0.744	0.736	0.728	0.713	0.699	0.685	0.672	0.659	0.647	0.635	0.624	0.608	0.592	0.577	0.563
60	0.707	0.697	0.687	0.678	0.668	0.659	0.651	0.634	0.617	0.602	0.587	0.573	0.560	0.547	0.535	0.517	0.501	0.486	0.471
70	0.640	0.629	0.618	0.607	0.597	0.587	0.578	0.559	0.542	0.526	0.510	0.496	0.482	0.469	0.456	0.439	0.423	0.408	0.394
80	0.576	0.564	0.553	0.542	0.531	0.521	0.511	0.492	0.474	0.458	0.442	0.428	0.414	0.402	0.390	0.373	0.358	0.344	0.330
90	0.517	0.505	0.493	0.482	0.471	0.461	0.451	0.432	0.415	0.399	0.384	0.370	0.357	0.345	0.334	0.319	0.304	0.292	0.280
100	0.464	0.451	0.440	0.428	0.418	0.408	0.398	0.380	0.363	0.348	0.334	0.321	0.309	0.298	0.288	0.274	0.261	0.249	0.239
110	0.416	0.404	0.392	0.381	0.371	0.361	0.352	0.335	0.319	0.305	0.292	0.281	0.270	0.259	0.250	0.237	0.226	0.215	0.206
120	0.373	0.361	0.350	0.340	0.330	0.321	0.313	0.297	0.282	0.269	0.257	0.246	0.236	0.227	0.219	0.207	0.197	0.187	0.179
130	0.336	0.325	0.314	0.305	0.295	0.287	0.279	0.264	0.251	0.239	0.228	0.218	0.209	0.200	0.193	0.182	0.173	0.164	0.157
140	0.303	0.292	0.283	0.274	0.265	0.257	0.250	0.236	0.224	0.213	0.203	0.194	0.185	0.178	0.171	0.161	0.153	0.145	0.138
150	0.274	0.264	0.255	0.247	0.239	0.231	0.224	0.212	0.201	0.190	0.181	0.173	0.165	0.159	0.152	0.144	0.136	0.129	0.123
160	0.249	0.240	0.231	0.223	0.216	0.209	0.203	0.191	0.181	0.171	0.163	0.155	0.149	0.142	0.137	0.129	0.122	0.116	0.110
170	0.227	0.218	0.210	0.203	0.196	0.190	0.184	0.173	0.164	0.155	0.147	0.140	0.134	0.128	0.123	0.116	0.110	0.104	0.099
180	0.207	0.199	0.192	0.185	0.179	0.173	0.167	0.157	0.149	0.141	0.134	0.127	0.122	0.116	0.111	0.105	0.099	0.094	0.089
190	0.190	0.183	0.176	0.169	0.164	0.158	0.153	0.144	0.136	0.128	0.122	0.116	0.111	0.106	0.101	0.095	0.090	0.085	0.081
200	0.175	0.168	0.162	0.156	0.150	0.145	0.140	0.132	0.124	0.118	0.112	0.106	0.101	0.097	0.093	0.087	0.082	0.078	0.074
210	0.161	0.155	0.149	0.143	0.138	0.134	0.129	0.121	0.114	0.108	0.102	0.097	0.093	0.089	0.085	0.080	0.075	0.071	0.068
220	0.149	0.143	0.138	0.133	0.128	0.123	0.119	0.112	0.105	0.100	0.094	0.090	0.086	0.082	0.078	0.074	0.069	0.066	0.062
230	0.138	0.133	0.128	0.123	0.118	0.114	0.110	0.104	0.097	0.092	0.087	0.083	0.079	0.075	0.072	0.068	0.064	0.061	0.058
240	0.129	0.123	0.119	0.114	0.110	0.106	0.103	0.096	0.090	0.085	0.081	0.077	0.073	0.070	0.067	0.063	0.059	0.056	0.053
250	0.120	0.115	0.110	0.106	0.102	0.099	0.095	0.089	0.084	0.079	0.075	0.071	0.068	0.065	0.062	0.058	0.055	0.052	0.049

Table 11: Design Compressive Stress, f_{cd} (MPa) For Column Flexural Buckling Class a

KL/r	**Yield Stress, f_y (MPa)**																		
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**	**420**	**450**	**480**	**510**	**540**
10	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
20	181.8	190.6	199.5	208.4	217.2	226.0	234.8	252.4	269.9	287.4	304.8	322.2	339.5	356.7	374.0	399.7	425.3	450.9	476.3
30	177.6	186.2	194.7	203.2	211.7	220.1	228.5	245.3	262.0	278.5	295.0	311.4	327.6	343.8	359.9	383.8	407.6	431.0	454.3
40	173.1	181.3	189.4	197.4	205.4	213.4	221.4	237.1	252.7	268.0	283.2	298.2	313.1	327.7	342.2	363.5	384.3	404.7	424.7
50	167.9	175.5	183.1	190.6	198.0	205.4	212.6	227.0	241.0	254.7	268.1	281.1	293.9	306.2	318.3	335.6	352.2	367.9	382.9
60	161.5	168.5	175.4	182.1	188.7	195.2	201.6	214.0	225.9	237.3	248.1	258.5	268.3	277.6	286.4	298.6	309.9	320.1	329.4
70	153.7	159.8	165.7	171.5	177.0	182.4	187.6	197.5	206.7	215.2	223.0	230.2	236.9	242.9	248.5	256.0	262.6	268.4	273.5
80	144.1	149.1	153.9	158.4	162.7	166.8	170.6	177.8	184.1	189.8	194.9	199.5	203.5	207.2	210.5	214.8	218.5	221.8	224.6
90	132.7	136.5	140.0	143.3	146.4	149.2	151.9	156.6	160.8	164.4	167.5	170.3	172.7	174.9	176.9	179.4	181.6	183.5	185.2
100	120.2	122.9	125.3	127.6	129.6	131.5	133.3	136.4	139.0	141.3	143.2	145.0	146.5	147.9	149.1	150.7	152.1	153.3	154.3
110	107.5	109.3	111.0	112.5	113.9	115.1	116.3	118.3	120.0	121.5	122.8	123.9	124.9	125.8	126.6	127.7	128.6	129.4	130.1
120	95.5	96.7	97.9	98.9	99.8	100.7	101.5	102.8	104.0	105.0	105.9	106.6	107.3	108.0	108.5	109.2	109.9	110.5	111.0
130	84.6	85.5	86.3	87.0	87.7	88.3	88.8	89.8	90.6	91.3	92.0	92.5	93.0	93.5	93.9	94.4	94.9	95.3	95.7
140	75.2	75.8	76.4	76.9	77.4	77.8	78.2	78.9	79.5	80.0	80.5	80.9	81.3	81.6	81.9	82.3	82.6	83.0	83.2
150	67.0	67.4	67.9	68.2	68.6	68.9	69.2	69.7	70.2	70.6	70.9	71.2	71.5	71.8	72.0	72.3	72.6	72.9	73.1
160	59.9	60.3	60.6	60.9	61.1	61.4	61.6	62.0	62.4	62.7	62.9	63.2	63.4	63.6	63.8	64.0	64.3	64.5	64.6
170	53.8	54.1	54.3	54.6	54.8	55.0	55.1	55.5	55.7	56.0	56.2	56.4	56.6	56.7	56.9	57.1	57.3	57.4	57.6
180	48.6	48.8	49.0	49.2	49.3	49.5	49.6	49.9	50.1	50.3	50.5	50.6	50.8	50.9	51.0	51.2	51.3	51.5	51.6
190	44.0	44.2	44.3	44.5	44.6	44.7	44.9	45.1	45.3	45.4	45.6	45.7	45.8	45.9	46.0	46.2	46.3	46.4	46.5
200	40.0	40.2	40.3	40.4	40.5	40.7	40.7	40.9	41.1	41.2	41.3	41.4	41.5	41.6	41.7	41.8	41.9	42.0	42.1
210	36.6	36.7	36.8	36.9	37.0	37.1	37.2	37.3	37.4	37.6	37.7	37.8	37.8	37.9	38.0	38.1	38.2	38.3	38.3
220	33.5	33.6	33.7	33.8	33.9	34.0	34.0	34.2	34.3	34.4	34.5	34.5	34.6	34.7	34.7	34.8	34.9	35.0	35.0
230	30.8	30.9	31.0	31.1	31.2	31.2	31.3	31.4	31.5	31.6	31.6	31.7	31.8	31.8	31.9	31.9	32.0	32.1	32.1
240	28.5	28.5	28.6	28.7	28.7	28.8	28.8	28.9	29.0	29.1	29.1	29.2	29.3	29.3	29.4	29.4	29.5	29.5	29.6
250	26.3	26.4	26.5	26.5	26.6	26.6	26.7	26.7	26.8	26.9	26.9	27.0	27.0	27.1	27.1	27.2	27.2	27.3	27.3

Table 12: Design Compressive Stress, f_{cd} (MPa) For Column Flexural Buckling Class b

KL/r	Yield Stress, f_y (MPa)																		
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**	**420**	**450**	**480**	**510**	**540**
10	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
20	181.7	190.5	199.2	207.9	216.6	225.3	233.9	251.1	268.2	285.2	302.2	319.1	335.9	352.6	369.3	394.1	418.8	443.3	467.7
30	175.2	183.4	191.6	199.7	207.9	215.9	224.0	240.0	255.8	271.5	287.0	302.4	317.7	332.8	347.8	370.0	391.9	413.6	434.9
40	168.2	175.8	183.4	191.0	198.4	205.8	213.2	227.7	242.0	256.0	269.8	283.4	296.8	309.9	322.7	341.6	360.0	377.8	395.2
50	160.5	167.5	174.3	181.1	187.8	194.4	200.9	213.6	226.0	238.0	249.7	261.0	272.0	282.6	292.8	307.6	321.6	334.8	347.4
60	151.8	158.0	164.0	169.9	175.6	181.3	186.8	197.4	207.5	217.1	226.3	235.0	243.3	251.1	258.5	268.9	278.5	287.3	295.5
70	142.0	147.2	152.2	157.1	161.8	166.4	170.7	179.1	186.8	194.0	200.6	206.8	212.6	217.9	222.9	229.7	235.8	241.3	246.2
80	131.1	135.3	139.3	143.1	146.8	150.2	153.5	159.6	165.2	170.3	174.8	179.0	182.8	186.3	189.5	193.9	197.7	201.1	204.2
90	119.5	122.7	125.8	128.6	131.3	133.8	136.2	140.6	144.4	147.9	151.0	153.8	156.3	158.6	160.7	163.5	165.9	168.1	170.1
100	107.8	110.2	112.4	114.5	116.4	118.2	119.9	123.0	125.6	128.0	130.1	132.0	133.7	135.2	136.6	138.5	140.2	141.7	143.0
110	96.5	98.3	99.9	101.4	102.8	104.1	105.3	107.4	109.3	111.0	112.4	113.8	115.0	116.0	117.0	118.3	119.5	120.5	121.5
120	86.2	87.5	88.6	89.7	90.7	91.7	92.5	94.1	95.4	96.6	97.7	98.6	99.5	100.3	101.0	101.9	102.8	103.6	104.2
130	76.9	77.8	78.7	79.5	80.3	81.0	81.6	82.7	83.7	84.6	85.4	86.1	86.8	87.3	87.9	88.6	89.2	89.8	90.3
140	68.7	69.4	70.1	70.7	71.3	71.8	72.3	73.1	73.9	74.6	75.2	75.7	76.2	76.6	77.1	77.6	78.1	78.5	78.9
150	61.6	62.1	62.6	63.1	63.6	64.0	64.3	65.0	65.6	66.1	66.6	67.0	67.4	67.7	68.1	68.5	68.9	69.2	69.5
160	55.4	55.8	56.2	56.6	56.9	57.3	57.5	58.1	58.5	59.0	59.3	59.7	60.0	60.3	60.5	60.9	61.2	61.5	61.7
170	50.0	50.3	50.7	51.0	51.2	51.5	51.7	52.2	52.5	52.9	53.2	53.5	53.7	53.9	54.1	54.4	54.7	54.9	55.1
180	45.3	45.6	45.9	46.1	46.3	46.5	46.7	47.1	47.4	47.7	47.9	48.1	48.3	48.5	48.7	48.9	49.2	49.3	49.5
190	41.2	41.5	41.7	41.9	42.1	42.2	42.4	42.7	42.9	43.2	43.4	43.6	43.7	43.9	44.0	44.2	44.4	44.6	44.7
200	37.6	37.8	38.0	38.2	38.3	38.5	38.6	38.9	39.1	39.3	39.5	39.6	39.8	39.9	40.0	40.2	40.3	40.5	40.6
210	34.5	34.7	34.8	35.0	35.1	35.2	35.3	35.5	35.7	35.9	36.0	36.2	36.3	36.4	36.5	36.6	36.8	36.9	37.0
220	31.7	31.9	32.0	32.1	32.2	32.3	32.4	32.6	32.8	32.9	33.0	33.1	33.2	33.3	33.4	33.6	33.7	33.8	33.9
230	29.2	29.4	29.5	29.6	29.7	29.8	29.9	30.0	30.1	30.3	30.4	30.5	30.6	30.7	30.7	30.8	30.9	31.0	31.1
240	27.1	27.2	27.3	27.3	27.4	27.5	27.6	27.7	27.8	27.9	28.0	28.1	28.2	28.3	28.3	28.4	28.5	28.6	28.7
250	25.1	25.2	25.3	25.3	25.4	25.5	25.6	25.7	25.8	25.9	26.0	26.0	26.1	26.2	26.2	26.3	26.4	26.5	26.5

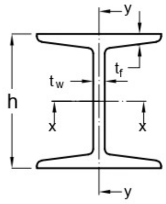
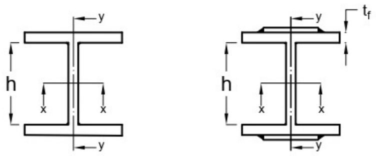
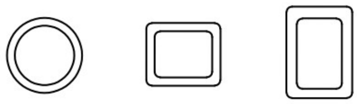
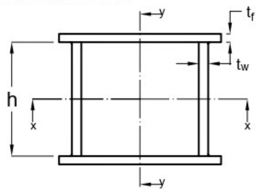
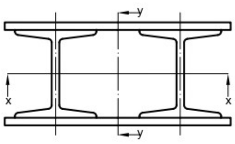
Table 13: Design Compressive Stress, f_{cd} (MPa) For Column Flexural Buckling Class c

KL/r	Yield Stress, f_y (MPa)																	
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**	**420**	**450**	**480**	**510**	**540**
10	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
20	181.7	190.3	198.9	207.4	215.9	224.4	232.8	249.6	266.3	282.8	299.3	315.6	331.8	348.0	364.0	387.9	411.5	435.0	458.2
30	172.4	180.3	188.1	195.9	203.6	211.3	219.0	234.1	249.0	263.8	278.4	292.8	307.0	321.1	334.9	355.5	375.6	395.4	414.9
40	162.9	170.0	177.1	184.1	191.0	197.8	204.6	217.9	231.0	243.7	256.2	268.4	280.4	292.1	303.6	320.3	336.5	352.2	367.4
50	152.9	159.2	165.5	171.6	177.6	183.5	189.3	200.6	211.5	222.1	232.3	242.2	251.7	260.9	269.7	282.5	294.5	305.9	316.7
60	142.4	147.8	153.1	158.2	163.3	168.2	173.0	182.2	190.9	199.2	207.1	214.6	221.7	228.5	234.9	243.9	252.3	260.0	267.2
70	131.2	135.7	140.1	144.3	148.3	152.2	156.0	163.2	169.8	176.1	181.9	187.3	192.3	197.1	201.5	207.7	213.3	218.4	223.1
80	119.8	123.4	126.8	130.1	133.3	136.3	139.2	144.5	149.4	154.0	158.1	161.9	165.5	168.7	171.8	175.9	179.7	183.1	186.1
90	108.4	111.2	113.9	116.4	118.8	121.0	123.2	127.1	130.7	133.9	136.9	139.6	142.0	144.3	146.4	149.3	151.8	154.1	156.2
100	97.5	99.6	101.7	103.6	105.3	107.0	108.6	111.5	114.1	116.5	118.6	120.5	122.3	123.9	125.4	127.4	129.2	130.8	132.3
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	99.8	101.5	103.1	104.5	105.8	106.9	108.0	109.5	110.8	112.0	113.1
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6
130	70.0	71.0	71.9	72.8	73.5	74.3	75.0	76.2	77.3	78.3	79.2	80.0	80.7	81.4	82.0	82.9	83.6	84.3	84.9
140	62.9	63.6	64.4	65.0	65.6	66.2	66.7	67.7	68.6	69.3	70.0	70.7	71.2	71.8	72.3	72.9	73.5	74.1	74.6
150	56.6	57.2	57.8	58.3	58.8	59.2	59.7	60.4	61.1	61.7	62.3	62.8	63.3	63.7	64.1	64.6	65.1	65.5	65.9
160	51.1	51.6	52.1	52.5	52.9	53.3	53.6	54.2	54.8	55.3	55.7	56.1	56.5	56.9	57.2	57.6	58.0	58.4	58.7
170	46.4	46.8	47.1	47.5	47.8	48.1	48.4	48.9	49.3	49.8	50.1	50.5	50.8	51.1	51.3	51.7	52.0	52.3	52.6
180	42.2	42.5	42.8	43.1	43.4	43.6	43.9	44.3	44.7	45.0	45.3	45.6	45.8	46.1	46.3	46.6	46.9	47.1	47.3
190	38.5	38.8	39.0	39.3	39.5	39.7	39.9	40.3	40.6	40.9	41.1	41.4	41.6	41.8	42.0	42.2	42.5	42.7	42.9
200	35.3	35.5	35.7	35.9	36.1	36.3	36.5	36.8	37.0	37.3	37.5	37.7	37.9	38.1	38.2	38.4	38.6	38.8	39.0
210	32.4	32.6	32.8	33.0	33.1	33.3	33.4	33.7	33.9	34.1	34.3	34.5	34.7	34.8	34.9	35.1	35.3	35.4	35.6
220	29.9	30.1	30.2	30.4	30.5	30.6	30.8	31.0	31.2	31.4	31.5	31.7	31.8	31.9	32.1	32.2	32.4	32.5	32.6
230	27.6	27.8	27.9	28.0	28.2	28.3	28.4	28.6	28.8	28.9	29.1	29.2	29.3	29.4	29.5	29.7	29.8	29.9	30.0
240	25.6	25.7	25.9	26.0	26.1	26.2	26.3	26.4	26.6	26.7	26.9	27.0	27.1	27.2	27.3	27.4	27.5	27.6	27.7
250	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.7	24.8	24.9	25.0	25.1	25.2	25.3	25.4	25.5	25.6	25.7

Table 14: Design Compressive Stress, f_{cd} (MPa) For Column Flexural Buckling Class b

KL/r	Yield Stress, f_y (MPa)																		
	200	**210**	**220**	**230**	**240**	**250**	**260**	**280**	**300**	**320**	**340**	**360**	**380**	**400**	**420**	**450**	**480**	**510**	**540**
10	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
20	181.6	190.0	198.2	206.5	214.7	222.8	230.9	247.0	262.8	278.6	294.1	309.6	324.8	340.0	354.9	377.2	399.1	420.8	442.2
30	167.6	175.0	182.2	189.4	196.5	203.6	210.5	224.3	237.9	251.2	264.2	277.1	289.7	302.2	314.4	332.4	350.0	367.1	383.9
40	154.3	160.6	166.9	173.0	179.1	185.1	191.0	202.6	213.9	224.8	235.5	245.9	256.0	265.8	275.4	289.4	302.8	315.7	328.2
50	141.3	146.7	152.0	157.1	162.2	167.2	172.1	181.5	190.6	199.3	207.7	215.8	223.6	231.1	238.3	248.6	258.4	267.7	276.5
60	128.6	133.1	137.5	141.7	145.9	149.9	153.8	161.3	168.4	175.2	181.6	187.7	193.5	199.0	204.2	211.7	218.6	225.1	231.2
70	116.4	120.1	123.6	127.0	130.3	133.5	136.5	142.4	147.8	152.9	157.7	162.3	166.5	170.5	174.3	179.6	184.5	189.0	193.2
80	104.8	107.8	110.6	113.3	115.8	118.3	120.7	125.2	129.3	133.2	136.7	140.1	143.2	146.1	148.8	152.6	156.1	159.3	162.2
90	94.1	96.4	98.6	100.7	102.8	104.7	106.5	109.9	113.1	116.0	118.6	121.1	123.4	125.5	127.6	130.3	132.8	135.2	137.3
100	84.3	86.2	87.9	89.6	91.1	92.6	94.0	96.7	99.1	101.3	103.3	105.2	106.9	108.5	110.0	112.1	113.9	115.7	117.2
110	75.6	77.0	78.4	79.7	81.0	82.1	83.2	85.3	87.1	88.8	90.4	91.8	93.1	94.4	95.5	97.1	98.5	99.8	101.0
120	67.8	69.0	70.1	71.1	72.1	73.0	73.9	75.5	77.0	78.3	79.5	80.6	81.7	82.6	83.5	84.7	85.8	86.9	87.8
130	61.0	62.0	62.8	63.7	64.5	65.2	65.9	67.2	68.3	69.4	70.4	71.2	72.1	72.8	73.5	74.5	75.4	76.2	76.9
140	55.0	55.8	56.5	57.2	57.8	58.4	59.0	60.0	61.0	61.8	62.6	63.3	64.0	64.6	65.2	66.0	66.7	67.3	67.9
150	49.8	50.4	51.0	51.6	52.1	52.6	53.1	53.9	54.7	55.4	56.0	56.6	57.2	57.7	58.1	58.8	59.3	59.9	60.4
160	45.2	45.7	46.2	46.7	47.1	47.5	47.9	48.6	49.3	49.9	50.4	50.9	51.3	51.7	52.1	52.7	53.1	53.6	54.0
170	41.2	41.6	42.1	42.4	42.8	43.1	43.5	44.1	44.6	45.1	45.5	45.9	46.3	46.7	47.0	47.4	47.8	48.2	48.6
180	37.7	38.0	38.4	38.7	39.0	39.3	39.6	40.1	40.5	41.0	41.3	41.7	42.0	42.3	42.6	43.0	43.3	43.6	43.9
190	34.5	34.9	35.2	35.4	35.7	35.9	36.2	36.6	37.0	37.4	37.7	38.0	38.2	38.5	38.7	39.1	39.4	39.6	39.9
200	31.8	32.0	32.3	32.5	32.8	33.0	33.2	33.6	33.9	34.2	34.5	34.7	35.0	35.2	35.4	35.7	35.9	36.2	36.4
210	29.3	29.6	29.8	30.0	30.2	30.4	30.5	30.9	31.2	31.4	31.7	31.9	32.1	32.3	32.5	32.7	32.9	33.1	33.3
220	27.1	27.3	27.5	27.7	27.9	28.0	28.2	28.5	28.7	29.0	29.2	29.4	29.6	29.7	29.9	30.1	30.3	30.5	30.6
230	25.2	25.3	25.5	25.7	25.8	26.0	26.1	26.4	26.6	26.8	27.0	27.1	27.3	27.5	27.6	27.8	27.9	28.1	28.2
240	23.4	23.6	23.7	23.9	24.0	24.1	24.2	24.5	24.7	24.8	25.0	25.2	25.3	25.4	25.5	25.7	25.9	26.0	26.1
250	21.8	22.0	22.1	22.2	22.3	22.5	22.6	22.8	22.9	23.1	23.2	23.4	23.5	23.6	23.7	23.9	24.0	24.1	24.2

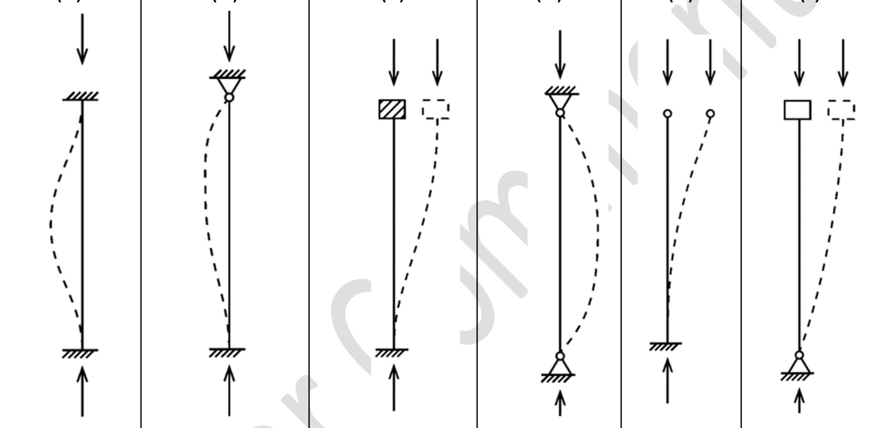
Table 15: Buckling class of cross-sections

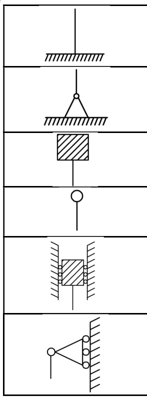
Cross- Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I- Sections 	$h/b_f > 1.2$: $t_f \le 40$ mm	x-x y-y	a b
	$40 \text{ mm} \le t_f \le 100$ mm	x-x y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	x-x y-y	b c
	$t_f > 100$ mm	x-x y-y	c d
Welded I- Section 	$t_f \le 40$ mm	x-x y-y	b c
	$t_f > 40$ mm	x-x y-y	c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$ $h/t_w < 30$	x-x y-y	c c
Channel, Angle, T and Solid Sections 		Any	c
Built-up Members 		Any	c

7.2 Effective Length of Compression Members

7.2.1 The effective length KL, is calculated from the actual length L, of the member, considering boundary conditions at the support. The actual length shall be taken as the center-to-center distance of its intersections with the supporting members in the plane of the buckling deformation. When the boundary conditions of the compression member can be assessed accurately, Annex F can be referred. When the boundary conditions in the plane of buckling can be assessed, the effective length, KL can be calculated on the basis of Table 16. Where frame analysis doesn't consider the equilibrium of a framed structure in the deformed shape (second-order analysis or advance analysis), the effective length of compression members in such cases can be calculated using the procedure given in Annex F

Table 16: Effective length of Prismatic Compression Members

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shaped of column is shown by dashed line						
Theoretical K Value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.8	1.0	1.2	2.0	2.0
End condition code						

	Rotation fixed and translation fixed
	Rotation free and translation fixed
	Rotation fixed and translation free
	Rotation free and translation free
	Rotation fixed, horizontal translation fixed, and vertical translation free
	Rotation free, horizontal translation fixed, and vertical translation free

7.2.2 Eccentric Beam Connections

When beam connections are eccentric in plan relative to the column axes, the same restraint conditions as for concentric connections are assumed to apply. This is valid if the connections extend across the flange or web of the columns, and the beam web is within or in direct contact with the column section. If practical difficulties prevent this, in non-sway frames, the effective length shall be considered as the distance between the points of restraint.

7.2.3 Compression Members in Trusses

For bolted, or welded trusses and braced frames, the effective length (KL) of compression members shall be taken as 0.7 to 1.0 times the distance between the centers of the connections, depending on the degree of end restraint. For truss members subject to buckling in the plane perpendicular to the truss plane, the effective length (KL) shall be taken as the distance between the centers of intersection. The design of angle struts shall follow the specifications in Section 7.4.

7.3 Design Details

7.3.1 Thickness of plate Elements

The classification of members based on the thickness of their constituent plate elements must meet the width-to-thickness ratio requirements outlined in Table 1.

7.3.2 Effective Sectional Area, A_e

The gross sectional area shall be considered the effective sectional area for all compression members fabricated by welding, bolting, provided the section is semi-compact or better. Holes not filled with bolts, or pins shall be deducted from the gross area to determine the effective sectional area.

7.3.3 Eccentricity for Columns

7.3.3.1 To determine the stress in a stanchion or column section, beam reactions or similar loads shall be assumed to be applied at an eccentricity of 100 mm from the section face or at the center of bearing, whichever is greater. The following exceptions apply:

a) For cap connections, the load shall be assumed to be applied at the face of the column or stanchion section, or at the edge of the packing (if used) towards the span of the beam.
b) For roof trusses bearing on a cap, no eccentricity shall be considered for simple bearings without connections capable of developing significant moments. For web member connections with the face, the actual eccentricity shall be considered.

7.3.3.2 In continuous columns, the bending moments caused by loading eccentricities at any floor can be equally divided between the columns above and below that floor level, provided the moment of inertia of one column section divided by its effective length does not exceed 1.5 times the corresponding value of the other column. If this ratio is exceeded, the bending moment shall be divided in proportion to the moment of inertia of the column sections divided by their respective effective lengths.

7.3.4 Splices

7.3.4.1 When the ends of compression members are prepared to bear over the entire area, they must be spliced to maintain the correct position of the connected members and to resist any bending or tension forces. These splices shall preserve the intended stiffness of the member along each axis and shall be placed as close to the point of inflection as possible. Otherwise, their capacity must be sufficient to carry the magnified moment. The ends of compression members designed for bearing shall always be machined to ensure perfect contact of the bearing surfaces.

7.3.4.2 In cases where compression members are not faced for full bearing, the splices must be engineered to effectively transmit all the forces acting on the members. Also whenever feasible, splices shall be designed and positioned to align the centroidal axis of the splice as closely as possible with the centroidal axes of the members being joined, thus minimizing eccentricity. However, if eccentricity exists in the joint, the resulting stress must be taken into consideration.

7.4 Angle Struts

The design strength of single angle members loaded in compression through one of its legs is affected by flexural torsional buckling and bending. The design compressive strength of such eccentricity loaded angle members, in-lieu of a more exact second order design under combined bending axial compression, may be evaluated as given below.

The design compressive strength, of single angles loaded through connection to one leg parallel to the a-a axis (a-a axis being either the 2-2 or the y-y axis, depending on which leg is connected to the connection:

$$f_{cde} = K_f \chi_{aa} \frac{f_y}{\gamma_{m0}}$$

where, $K_f = k_1 + k_2\lambda_{aa} + k_3\lambda_{\phi}$; and

χ_{aa}= Stress reduction factor for buckling class 'b', using the non-dimensional effective slenderness ratio, λ_{aa}.

K_f= modification factor to account for eccentric end connections; and

k_1, k_2, k_3= constants depending upon the end condition (Table 17)

$$\lambda_{vv} = \frac{\frac{l_{aa}}{r_{aa}}}{\epsilon\sqrt{\frac{\pi^2 E}{230}}}$$

$$\lambda_{\phi} = \frac{(b_1 + b_2)/2t}{\epsilon\sqrt{\frac{\pi^2 E}{230}}}$$

l_{aa} =center-to-center length of the lateral support preventing translation of the member perpendicular to a-a axis (a-a axis being either the z-z or the y-y axis, depending on which leg is connected to the gusset);

r_{aa} =radius of gyration of the angle member about the a-a axis (a-a axis being either the z-z or the y y axis, depending on which leg is connected to the gusset), parallel to the plane of the end gusset plates;

b_1, b_2 =width of the connected and outstanding legs of the angle, respectively

t = thickness of the leg; ϵ = yield stress ratio $\left(\frac{230}{f_y}\right)^{0.5}$

Table 17: Values of k_1, k_2 and k_3

SN	End Connection	Gusset/Connecting Member Fixity	k_1	k_2	k_3
i	Fully welded or connected with two or more bolts	Fixed	0.798	0.563	-2.072
		Hinged	0.401	0.420	-1.040
ii	Single bolt	Fixed	0.418	0.547	-1.400
		Hinged	0.374	0.415	-2.072

Note: In-plane rotational restraint provided to the gusset by the supporting member. For partial restraint, K_f can be interpolated between the results for fixed and hinged cases

7.4.1 Double Angle Struts

7.4.1.1 When two discontinuous angle struts are connected back-to-back, on opposite sides of the gusset or a section, by not less than two bolts in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, KL, in the plane of the end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of restraints provided. The effective length, KL, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between the centers of intersections. The compressive stress capacity may be computed as per 7.1.1.

7.4.1.2 When such two angle discontinuous struts are connected back-to-back, to one side of a gusset or section by one or more bolts or by welding, it shall be designed according to 7.4 and requirements in 7.8 must be fulfilled.

7.4.2 Continuous members

Double angle continuous struts, which are components like flanges, chords, or ties in trusses or trussed girders, or the legs of towers, shall be designed as axially loaded compression members. Their effective length shall be determined according to the guidelines provided in section 7.2.3.

7.4.3 Combined Stresses

In addition to axial loads, if the struts carry loads which cause transverse bending, the combined bending and axial stresses shall be checked according to (biaxial chapter-yet to be written). For determining the permissible axial and bending stresses, the effective length shall be taken in accordance with 7.2 and 8.1.1.

7.5 Column Bases

7.5.1 Column bases shall have sufficient size, stiffness and strength to transmit the axial forces, bending moments and shear forces in columns to their foundation without exceeding the load carrying capacity of the supports. Anchor bolts and shear keys shall be provided wherever necessary. Shear resistance at the proper contact surface between steel base and concrete/grout may be calculated using a friction coefficient of 0.45.

7.5.2 The nominal bearing pressure between the base plate and the support below may be determined based on a linearly varying distribution of pressure. The maximum bearing pressure shall not exceed the bearing strength, which is equal to $0.45f_{ck}$, where f_{ck} is the smaller of the characteristic cube strength of concrete or the bedding material, whichever is less.

7.5.3 Slab Bases

Columns with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection,

7.5.4 Thickness of flexible base plate (Effective Area Method)

If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection c of the base plate beyond the face of the column and gusset may be taken as effective in transferring the column load as given in Figure 12, such that bearing pressure on the effective area does not exceed bearing capacity of concrete base.

The minimum thickness, t_s, of column bases under axial compression shall be:

$$\mathrm{t_s} = \sqrt{\frac{2.5\ \mathrm{w}\ \mathrm{c}^2 \gamma_{m0}}{\mathrm{f_y}}} > \mathrm{t_f}$$

7.5.5 Thickness of rigid base plate (Cantilever Method):

7.5.5 Thickness of rigid base plate (Cantilever Method):

The minimum thickness, t_s, of rectangular slab bases, supporting columns under axial compression shall be:

$$t_s = \sqrt{\frac{2.5\ w\ (a^2 - 0.3\ b^2)\gamma_{m0}}{f_y}} > t_f$$

7.5.5.1 When the slab does not distribute the column load uniformly, due to eccentricity of the load, special calculation shall be made to show that the base is adequate to resist the moment due to the non-uniform pressure from below.

7.5.5.2 Bases for bearing upon concrete or masonry need not be machined on the underside

7.5.5.3 In cases where the cap or base is fillet welded directly to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to transmit the design force. Where full strength butt welds are provided, machining of contact surfaces is not required

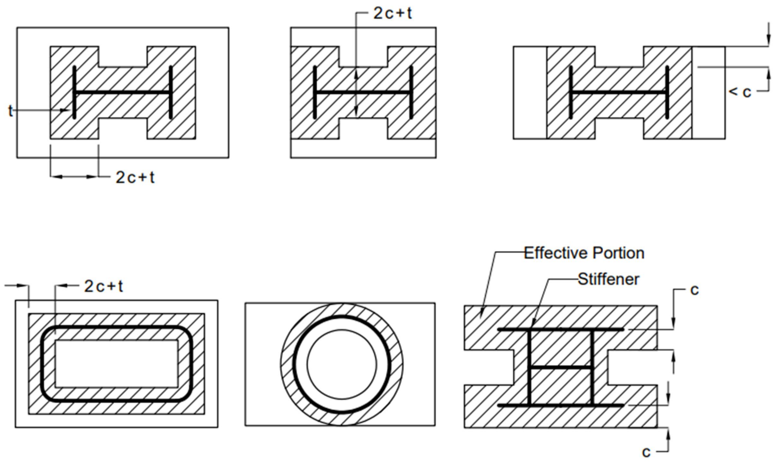


Figure 12: Effective Area of a Base Plate

7.5.6 Anchor bolted base plate connection

Anchor bolted base plate connections at column bases shall be designed to prevent the following:

a) Bearing failure of concrete under compression
b) Pullout cone failure of concrete due to tensile force in anchor bolts
c) Side face blowout failure of concrete due to tensile force in anchor bolts with headed or hooked ends
d) Wedge-cone failure of concrete due to shear force in anchor bolts, and
e) Bolt-concrete bond slip failure.

7.6 Laced Compression Members

7.6.1 General

7.6.1.1 Members comprising two main components laced and tied, shall where practicably have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis parallel to the plane of lacing. (See Figure 13A and B)

7.6.1.2 As far as practicable the lacing system shall be uniform throughout the length of the column.

7.6.1.3 Except for tie plates as specified in 7.6, double laced systems (see Figure 13B) and single laced systems (see Figure 13A) on opposite sides of the main components shall not be combined with cross members (ties) perpendicular to the longitudinal axis of the strut (See Figure 13C), unless all forces resulting from deformation of the strut members are calculated and provided for in the design of lacing and its fastenings.

7.6.1.4 Single laced systems, on opposite faces of the components being laced together shall preferably be in the same direction so that one is the shadow of the other, instead of being mutually opposed in direction.

7.6.1.5 The effective slenderness ratio, $(KL/r)_e$, of laced columns shall be taken as 1.05 times the $(KL/r)_0$, the actual maximum slenderness ratio, in order to account for shear deformation effects.

7.6.2 Width of Lacing Bars: In bolted construction, the minimum width of lacing bars shall be three times the nominal diameter of the end bolt.

7.6.3 Thickness of Lacing Bars: The thickness of lacing bars shall not be less than one-fortieth of its effective length for single lacing and one-sixtieth of the effective length for double lacing

Rolled sections or tubes of equivalent strength may be permitted instead of flats for lacing

7.6.4 Lacing Angle: Lacing bars, whether in double or single systems, shall be inclined at an angle not less than $40°$ nor more than $70°$ to the axis of the built-up member.

7.6.5 Spacing: The maximum spacing of lacing bars, whether connected by bolting or welding, shall also be such that the maximum slenderness ratio of the components of the main member (a_1/r_1), between consecutive lacing connections is not greater than 50 or 0.7 times the most unfavorable slenderness ratio of the member as a whole, whichever is less, where a_1 is the unsupported length of the individual member between lacing points, and r_1 is the minimum radius of gyration of the individual member being laced together.

Where lacing bars are not lapped to form the connection to the components of the members, they shall be connected that there is no appreciable change in the triangulation of the system.

7.6.6 Design of Lacing

7.6.6.1 The lacing shall be proportioned to resist a total transverse shear V_t, at any point in the member, equal to at least 2.5% of the axial force in the member and shall be divided equally among all transverse lacing systems in parallel planes.

7.6.6.2 For members carrying calculated bending stress due to eccentricity of loading, applied end moments and/or lateral loading, the lacing shall be proportioned to resist the actual shear due to bending, in addition to that specified in 7.6.6.1.

7.6.6.3 Slenderness Ratio of a Lacing Element

The slenderness ratio, KL/r of the lacing bars shall not exceed 140.

In bolted construction, the effective length of lacing bars for the determination of the design strength shall be taken as the length between the inner end fastener of the bars for single lacing, and as 0.7 of this length for double lacings effectively connected at intersections. In welded construction, the effective length shall be taken as 0.7 times the distance between the inner ends of welds connecting the single lacing bars to the members.

Note: The required section for lacing bars compression/tension members shall be determined by using the approximate design stress, f_{cd} subject to the requirements given in 7.6.3 to 7.6.6 and T_d in 6.1.

7.6.7 Attachment to main members:

The bolting or welding of lacing bars to the main members shall be sufficient to transmit the force calculated in the bars. Where welded lacing bars overlap the main members, the amount of lap measured along either edge of the lacing bar shall be not less than four times the thickness of the bar or the thickness of the element of the members to which it is connected, whichever is less. The welding shall be sufficient to transmit the load in the bar and shall, in any case, be provided along each side of the bar for the full length of lap.

Double lacing bars shall be jointed at intersections.

7.6.8 End tie plates: Laced compression members shall be provided with tie plates as per 7.7 at the ends of lacing systems and at intersection with other members/stays and at points where the lacing systems are interrupted.

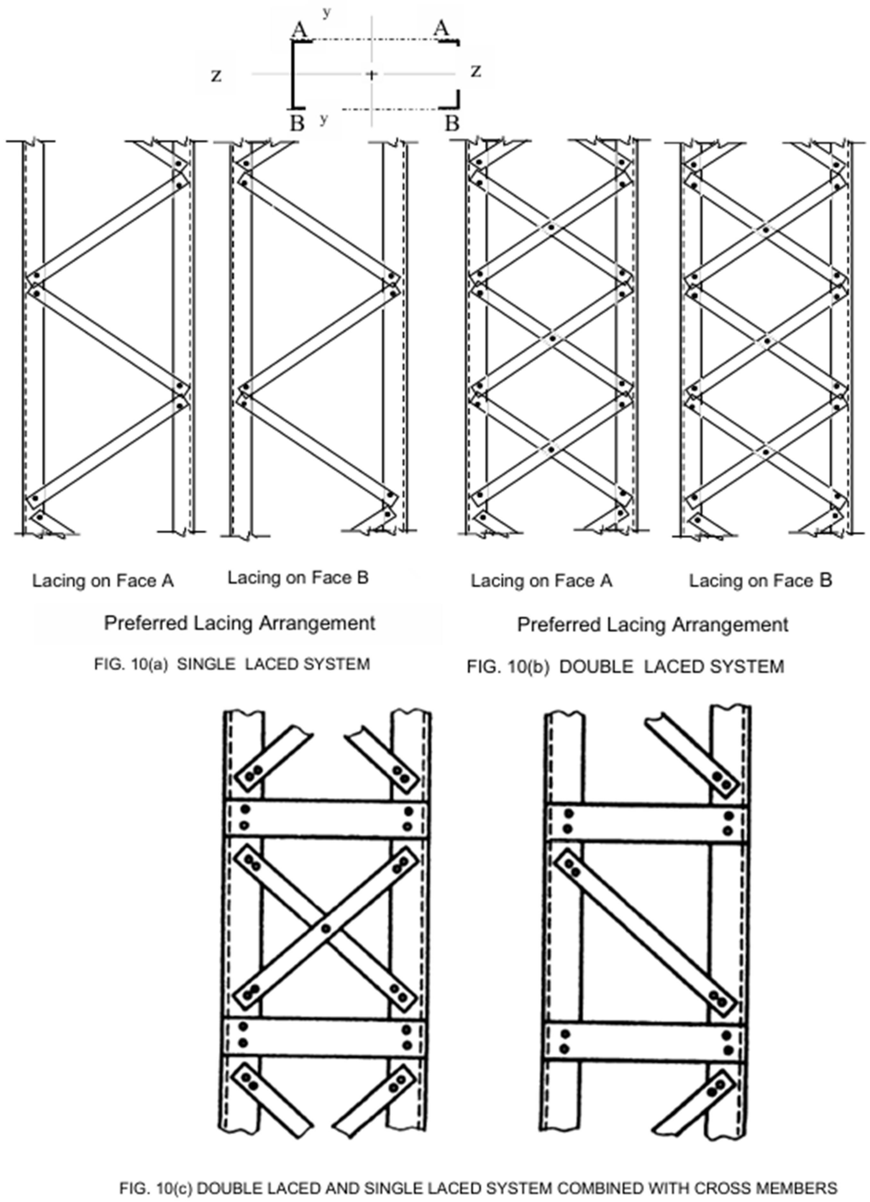


Figure 13: Laced Columns

7.7 Battened Compression members

7.7.1 General

7.7.1.1 Compression members composed of two main components battened individual shall preferably have the, members of the same cross-section and symmetrically disposed about their major axis. Where practicable, the compression members shall have a radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis parallel to the plane of the batten (See Figure 14).

7.7.1.2 Battened compression members, not complying with the requirements specified in this section or those subjected to eccentricity of loading, applied moments or lateral forces in the plane of the battens (figure yet to be drawn), shall be designed according to the exact theory of elastic stability or empirically, based on verification by tests.

Note: If the column section is subjected to eccentricity or other moments about an axis perpendicular to battens, the battens and the column section shall be specially designed for such moments and shears.

7.7.1.3 The battens shall be placed opposite to each other at each end of the member and at points where the member is stayed in its length and as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length from center-center of end connections.

7.7.1.4 The effective slenderness ratio $(KL/r)_e$ of battened columns, shall be taken as 1.1 times the $(KL/r)_0$, the maximum actual slenderness ratio of the column, to account for shear deformation effects.

7.7.2 Design of Battens

7.7.2.1 Battens

Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force V_t equal to 2.5 percent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens. Battened member carrying calculated bending moment due to eccentricity of axial loading, calculated end moments or lateral loads parallel to the plane of battens, shall be designed to carry actual shear in addition to the above shear. The main members shall also be checked for the same shear force and bending moments as for the battens.

Battens shall be of plates, angles, channels, or I-sections and at their ends shall be bolted or welded to the main components so as to resist simultaneously a shear $V_b = V_t C/NS$ along the column axis and a moment $M = V_t C/2N$ at each connection,

where, V_t = transverse shear force as defined above;

c= distance between center-to-center of battens, longitudinally;

N= number of parallel planes of battens; and

S=minimum transverse distance between the centroid of the bolt group/welding connecting the batten to the main members

7.7.2.2 Tie plates

Tie plates are members provided at the ends of battened and laced members, and shall be designed by the same method as battens. In no case shall a tie plate and its fastenings be incapable of carrying the forces for which the lacing or batten has been designed.

7.7.2.3 Size

When plates are used for battens, the end battens and those at points where the member is stayed in its length shall have an effective depth, longitudinally, not less than the perpendicular distance between the centroids of the main members. The intermediate battens shall have an effective depth of not less than three quarters of this distance, but in no case shall the effective depth of any batten be less than twice the width of one member, in the plane of the battens. The effective depth of a batten shall be taken as the longitudinal distance between outermost bolts or welds at the ends. The thickness of batten or the tie plates shall be not less than one-fiftieth of the distance between the innermost connecting lines of bolts or welds, perpendicular to the main member.

7.7.2.4 The requirement of bolt size and thickness of batten specified above does not apply when angles, channels or I-sections are used for battens with their legs or flanges perpendicular to the main member. However, it shall be ensured that the ends of the compression members are tied to achieve adequate rigidity.

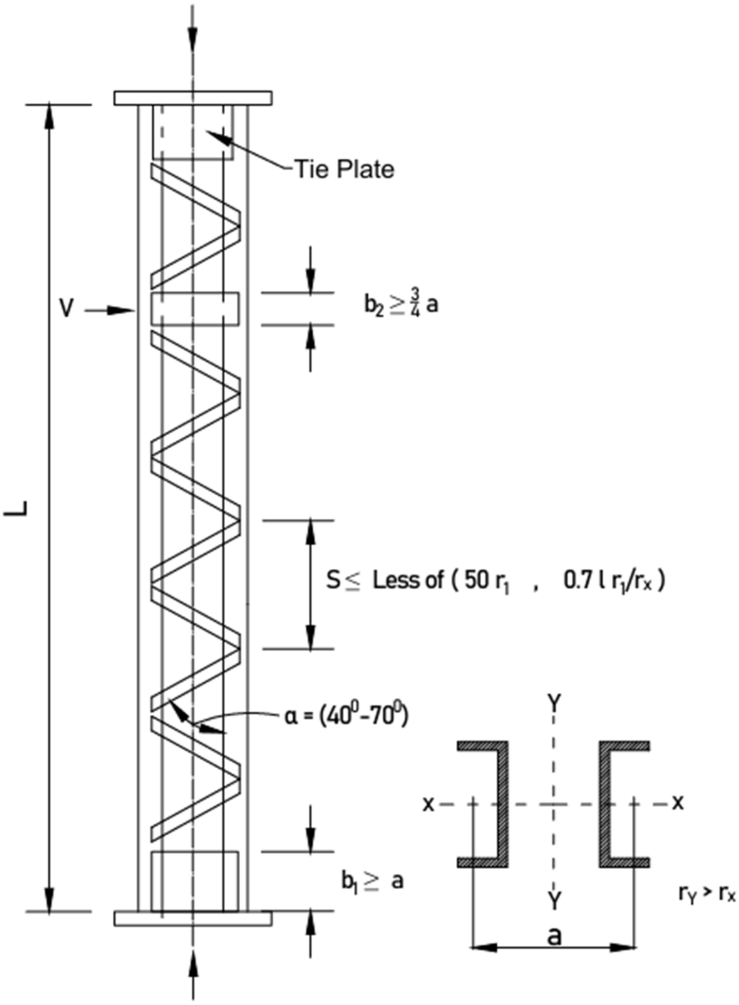


Figure 14: Lacing Specifications

7.7.3 Spacing of Battens:

In battened compression members where the individual members are not specifically checked for shear stress and bending moments, the spacing of battens, center to-center of its end fastenings, shall be such that the slenderness ratio (KL/r) of any component over that distance shall be neither greater than 50 nor greater than 0.7 times the slenderness ratio of the member as a whole about its z-z (axis parallel to the battens).

7.7.4 Attachment to Main Members

7.7.4.1 Welded connections:

Where tie or batten plates overlap the main members, the amount of lap shall be not less than four times the thickness of the plate. The length of weld connecting each edge of the batten plate to the member shall, in aggregate, be not less than half the depth of the batten plate. At least one-third of the weld shall be placed at each end of this edge. The length of weld and depth of batten plate shall be measured along the longitudinal axis of the main member.

In addition, the welding shall be returned along the other two edges of the plates transversely to the axis of the main member for a length not less than the minimum lap specified above.

7.8 Compression Members Composed of Two Components Back-to-back

7.8.1 Compression members composed of two angles, channels, or tees back-to-back in contact or separated by a small distance, shall be connected together by bolting or welding so that the ratio of most unfavorable slenderness of each member between the intermediate connections is not greater than 40 or 0.6 times the most unfavorable ratio of slenderness of the strut as a whole, whichever is less (See Section 10.)

7.8.2 In no case shall the ends of the strut be connected together with less than two bolts or their equivalent in welding, and there shall be not less than two additional connections equidistant in between, along the length of the strut. Where the members are separated back-to-back, the bolts through these connections shall pass through solid washers or packing in between. Where the legs of the connected angles or the connected tees are 125 mm wide or more, or where webs of channels are 150 mm wide or over, not less than two bolts shall be used in each connection, one on line of each gauge mark.

7.8.3 Where these connections are made by welding, solid packing shall be used to affect the jointing unless the members are sufficiently close together to permit direct welding, and the members shall be connected by welding along both pairs of edges of the main components.

7.8.4 The bolts or welds in these connections shall be sufficient to carry the shear force and moments, if any, specified for battened struts (see 7.7.3), and in no case shall the bolts be less than 16 mm diameter for members up to and including 10 mm thick; 20 mm diameter for members up to and including 16 mm thick; and 22 mm diameter for members over 16 mm thick.

Compression members connected by such bolting or welding shall not be subjected to transverse to the loading in a plane perpendicular bolted or welded surface.

7.8.5 Where the components are in contact back-to-back, the spacing of the bolts or intermittent welds shall not exceed the maximum spacing for compression members.

8 Design for Flexure

8.1 General

A member experiencing flexure shall have enough capacity to resist bending moment and shear forces while satisfying serviceability criteria as presented in Section 5. Members subjected to other forces in combination with bending moments shall be designed in accordance with Section 9.

8.1.1 Effective span of Beams

The effective span of a beam shall be taken as the distance between the centres of the supports, except where the point of application of the reaction is taken as eccentric at the support, when it shall be permissible to take the effective span as the length between the assumed lines of the reactions.

8.2 Design Strength in Bending (Flexure)

The design bending strength of beam, adequately supported against lateral torsional buckling (laterally supported beam) is governed by the yield stress (see 8.2.1). When a beam is not adequately supported against lateral buckling (laterally un-supported beams) the design bending strength may be governed by lateral torsional buckling strength (see 8.2.2).

The factored design moment, M at any section, in a beam due to external actions, shall satisfy

$$M \leq M_d$$

where, M_d = design bending strength of the section, calculated as given in 8.2.1.2.

8.2.1 Laterally supported Beam

A beam may be assumed to be adequately supported at the supports, provided the compression flange has full lateral restraint and nominal torsional restraint at supports supplied by web cleats, partial depth end plates, fin plates or continuity with the adjacent span. Full lateral restraint to compression flange may be assumed to exist if the frictional or other positive restraint of a floor connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the floor construction is capable of resisting this lateral force.

The design bending strength of a section which are not susceptible to web buckling under shear before yielding (where $d/t_w \leq 67\,\epsilon$) shall be determined according to 8.2.1.2.

8.2.1.1 Section with webs susceptible to shear buckling before yielding

When the flanges are plastic, compact or semi-compact but the web is susceptible to shear buckling before yielding ($d/t_w \leq 67\epsilon$), the design bending strength shall be calculated using one of the following methods:

a) The bending moment and axial force acting on the section may be assumed to be resisted by flanges only and the web is designed only to resist shear (see 8.4).
b) The bending moment and axial force acting on the section may be assumed to be resisted by the whole section. In such a case, the web shall be designed for combined shear and normal stresses using simple elastic theory in case of semi compact webs and simple plastic theory in the case of compact and plastic webs.

8.2.1.2 When the factored design shear force does not exceed $0.6V_d$, where V_d is the design shear strength of the cross section (see 8.4), the design bending strength, M_d shall be taken as:

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2 Z_e f_y/\gamma_{m0}$ in case of simply supported and $1.5\, Z_e f_y/\gamma_{m0}$ in case of cantilever beams;

where, $\beta_b = 1.0$ for plastic and compact sections;

$\beta_b = Z_e/Z_p$ for semi-compact sections;

$Z_p, Z_e =$ plastic and elastic section moduli of the cross section, respectively;

$f_y =$ yield stress of the material; $\gamma_{m0} =$ partial safety factor (see 5.4).

8.2.1.3 When the design shear force (factored), V exceeds $0.6V_d$, where V_d is the design shear strength of the cross section (see 8.4) the design bending strength, M_d shall be taken as:

$$M_d = M_{dv}$$

where, $M_{dv} =$ design bending strength under high shear as defined in 9.2.

8.2.1.4 Holes in the tension zone

a) The effect of holes in the tension flange, on the design bending strength need not be considered if:

$$A_{nf}/A_{gf} \geq (f_y/f_u)\,(\gamma_{m1}/\gamma_{m0})\,/0.9$$

where, $A_{nf}/A_{gf} =$ ratio of net to gross area of the flange in tension;

$f_y/f_u =$ ratio of yield and ultimate stress of the material;

$\gamma_{m1}/\gamma_{m0} =$ ratio of partial safety factors against ultimate to yield stress (see 5.4).

When the A_{nf}/A_{gf} does not satisfy the above requirement, the reduced effective flange area, A_{ef} satisfying the above equation may be taken as the effective flange area in tension, instead of A_{gf}.

b) The effect of holes in the tension region of the web on the design flexural strength need not be considered, if the limit given in (a) above is satisfied for the complete tension zone of the cross section, comprising the tension flange and tension region of the web.

c) Fastener holes in the compression zone of the cross section need not be considered in design bending strength calculation, except for oversize and slotted holes or holes without any fastener.

8.2.1.5 Shear lag effects

The shear lag effects in flanges may be disregarded provided:

a) For outstand elements (supported along one edge), $b_o \leq L_o / 20$; and

b) For internal elements (supported along two edges), $b_i \leq L_o / 10$.

where, $L_o =$ length between points of zero moment (inflection) in the span;

$b_o =$ width of the flange with outstand; $b_i =$ width of the flange as an internal element

Where these limits are exceeded, the effective width of flange for design strength may be calculated using specialist literature, or conservatively taken as the value satisfying the limit given above.

8.2.2 Laterally Unsupported Beams

Resistance to lateral torsional buckling need not be checked separately (member may be treated as laterally supported, see 8.2.1) in the following cases:

a) Bending is about the minor axis of the section,

b) Section is hollow (rectangular/ tubular) or solid bars, and

c) In case of major axis bending, λ_{LT} (as defined herein) is less than 0.4.

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by: $M_d = \beta_b Z_p f_{bd}$

where, $f_{bd} = \chi_{LT} f_y / \gamma_{m0}$

$\beta_b = 1.0$ for plastic and compact sections;

$= Z_e/Z_p$ for semi-compact sections;

$Z_p, Z_e =$ plastic section modulus and elastic section modulus with respect to extreme compression fibre;

$\chi_{LT} =$ bending stress reduction factor to account for lateral torsional buckling, in doubly symmetrical sections with lateral support at the ends given by:

$$\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \lambda_{LT}^2)^{0.5}} \leq 1.0$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

$\alpha_{LT} =$ the imperfection parameter =0.21 for rolled steel section and 0.49 for welded steel section

The non-dimensional slenderness ratio, λ_{LT}, is given by $\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} = \sqrt{\frac{f_y}{f_{cr,b}}} \leq \sqrt{\frac{1.2 Z_e f_y}{M_{cr}}}$

where, $M_{cr} =$ elastic lateral torsional buckling moment calculated in accordance with 8.2.2.1, and

$f_{cr,b} =$ extreme fibre bending compressive stress corresponding to elastic lateral torsional buckling moment (see 8.2.2.1)

8.2.2.1 Elastic lateral torsional buckling moment

In case of simply supported, prismatic members with symmetric cross section, the elastic lateral buckling moment, M_{cr}, can be determined from:

$$M_{cr} = \sqrt{\left[\frac{\pi^2 E I_y}{L_{LT}^2}\right]\left[G I_t + \frac{\pi^2 E I_w}{L_{LT}^2}\right]} = \beta_b Z_p f_{cr,b}$$

$f_{cr,b}$ of non-slender rolled steel sections in the above equation maybe approximately calculated from the following equations:

$$f_{cr,b} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

The following simplified equation maybe used in case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections, for calculating the elastic lateral buckling moment, M_{cr}:

$$M_{cr} = \frac{\pi^2 E I_y h_f}{2L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

where, I_t = torsional constant = $\sum b_i t_i^3/3$ for open section; I_w = warping constant;

I_y, r_y = moment of inertia and radius of gyration, respectively about the weaker axis;

L_{LT} = effective length for lateral torsional buckling (see 8.3);

h_f = centre-to-center distance between the flanges; t_f = thickness of the flange

The above equations for calculating M_{cr} may be conservatively used for channel section

Table 18: Effective length for Simply supported beams, L_{LT}

SN	Torsional Restraint	Warping restraint	Normal loading	Destabilizing loading
1	Full restrained	Both flanges partially restrained	$0.7L$	$0.85L$
2		Only Compression flange fully restrained	$0.75L$	$0.9L$
3		Both flanges fully restrained	$0.8L$	$0.95L$
4		Only Compression flange partially restrained	$0.85L$	$1.0L$
5		No Warping restrained in both flanges	$1.0L$	$1.2L$
6	Partially restrained by bottom flange support connection	No Warping restrained in both flanges	$1.0L + 2D$	$1.2L + 2D$
7		No Warping restrained in both flanges	$1.2L + 2D$	$1.4L + 2D$

1) Torsional restraint prevents rotation about the longitudinal axis
2) Warping restraint prevents rotation of the flange in its plane
3) D is the overall depth of the beam
4) In case of continuous beam, L shall be taken as the distance between points of inflection, and the restraint conditions at the points of inflection shall be considered.

8.3 Effective length for Lateral-Torsional Buckling

8.3.1 In simply supported beams with intermediate lateral restraints against lateral torsional buckling, the effective length for lateral torsional buckling, L_{LT} shall be taken as the length of the relevant segment in between the lateral restraints. In the case of intermediate partial lateral restraints, the effective length, L_{LT} shall be taken as equal to 1.2 times the length of the relevant segment in between the partial lateral restraints.

Restraints against torsional rotation at supports can be ensured through:

a) Web or flange cleats
b) Bearing stiffeners
c) External supports or lateral end frames providing lateral restraint to the compression flanges at the end
d) Them being built into the wall

8.3.2 For beams, which are provided with members giving effective lateral restraint to the compression flange at intervals along the span, in addition to the end torsional restraint required in 8.3.1, the effective length for lateral torsional buckling shall be taken as the distance, centre-to-centre of the restraint members in the relevant segment under normal loading condition and 1.2 times this distance, where the load is not acting on the beam at the shear and is acting towards the shear centre so as to have destabilizing effect during lateral torsional buckling deformation

8.3.3 For cantilever beams of projecting length L, the effective length L_{LT} to be used in 8.2.2.1 shall be taken as in Table 18 for different support conditions.

8.3.4 Where a member is provided intermediate lateral supports to improve the lateral buckling strength, these restraints should have sufficient strength and stiffness to prevent lateral movement of the compression flange at that point, relative to the end supports. The intermediate lateral buckling restraints should be either connected to an appropriate bracing system capable of transferring the restraint force to the effective lateral support at the ends of the member, or should be connected to an independent robust part of the structure capable of transferring the restraint force. Two or more parallel member requiring such lateral restraint shall not be simply connected together assuming mutual dependence for the lateral restraint.

The intermediate lateral restraints should be connected to the member as close to the compression flange as practicable. Such restraints should be closer to the shear centre of the compression flange than to the shear centre of the section. However, if torsional restraint preventing relative rotation between the two flanges is provided, the intermediate lateral restraint may be connected at any appropriate level.

For beams which are provided with members giving effective lateral restraint at intervals along the span, the effective lateral restraint shall be capable of resisting a force of 2.5 percent of the maximum force in the compression flange taken as divided equally between the points at which the restraint members are provided. Further, each restraint point should be capable of resisting 1 percent of the maximum force in the compression flange.

8.3.4.1 In a series of such beams, with solid webs, which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum flange force in one beam only.

8.3.4.2 In the case of a series of latticed beams, girders or roof trusses which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum force in the compression flange plus 1.25 percent of this force for every member of the series other than the first, up to a maximum total of 7.5 percent.

8.3.5 Purlins adequately restrained by sheeting need not be normally checked for the restraining forces required by rafters, roof trusses or portal frames that carry predominately roof loads provided there is bracing of adequate stiffness in the plane of rafters or roof sheeting which is capable of acting as a stressed skin diaphragm.

8.3.6 In case of beams with double curvature bending, adequate direct lateral support to the compression flange in the hogging moment region maybe provided as given above for simply supported beam. The effect of support to the tension (top) flange in the hogging moment region on lateral restraint to the compression flange may be considered as per specialist literature.

8.4 Shear

The factored design shear force, V, in a beam due to external actions shall satisfy:

$$V \leq V_d$$

where, V_d = design strength = V_n/γ_{m0}; γ_{m0} = partial safety factor against shear failure

The nominal shear strength of a cross-section, V_n, may be governed by plastic shear resistance (see 8.4.2) or strength of the web as governed by shear buckling (see).

The nominal plastic shear resistance under pure shear is given by: $V_n = V_p$

where, $V_p = \frac{f_{yw} A_v}{\sqrt{3}}$

f_{yw} = yield stress of the web;

A_v = Shear area

8.4.1.1 The shear area may be calculated as given below:

a) I-and Channel sections:
   a. Major Axis Bending:
      i. Hot-Rolled : $h.t_w$
      ii. Welding : $d.t_w$
   b. Minor Axis Bending:
      i. Hot rolled or welded : $2b.t_f$
b) Rectangular hollow sections of uniform thickness:
   a. Loaded parallel to depth (h) : $A.h/(b+h)$
   b. Loaded parallel to width (b) : $A.b/(b+h)$
c) Circular hollow tubes of uniform thickness : $2A/\pi$
d) Plates and solid bars : A

where, A = cross section area,

b = overall breadth of tubular section, breadth of I-section flanges

d = clear depth of the web between flanges; h = overall depth of the section

t_f = thickness of the flange; t_w = thickness of the web

Note: Fastener holes need not be accounted for in plastic design shear strength calculation provided that: $A_{vn} \geq (f_y/f_u)(\gamma_{m1}/\gamma_{m0})A_v/0.9$
If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Verification then needs to be performed for block shear failure.

8.4.2 Resistance to Shear Buckling

8.4.2.1 Resistance to shear buckling shall be verified as specified, when

$\frac{d}{t_w} \geq 67\epsilon_w$ for a web without stiffeners, and $> 67\epsilon_w \left(\frac{K_v}{5.35}\right)^{0.5}$ for a web with stiffeners where, K_v = shear buckling coefficient (see 8.4.2.2), and $\epsilon_w = \sqrt{\frac{230}{f_y}}$

8.4.2.2 Shear buckling design methods

The nominal shear strength, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods:

a) Simple post-critical method: The simple post critical method, based on the shear buckling strength can be used for webs of I section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. nominal shear strength is given by: $V_n = V_{cr}$
where, V_{cr} = shear force corresponding to web buckling $= A_v \tau_b$

τ_b = shear stress corresponding to web buckling determined as:

a) $\tau_b = \frac{f_{yw}}{\sqrt{3}}$, when $\gamma_w \le 0.8$

b) $\tau_b = [1 - 0.8(\lambda_w - 0.8)]\frac{f_{yw}}{\sqrt{3}}$, when $0.8 < \gamma_w < 1.2$

c) $\tau_b = \frac{f_{yw}}{\sqrt{3}\,\lambda_w^2}$, when $\gamma_w \ge 1.2$

where, λ_w = non-dimensional web slenderness ratio for shear buckling stress, given by:

$$\lambda_w = \sqrt{\left(\frac{f_{yw}}{\sqrt{3}\tau_{cr,e}}\right)}$$

$\tau_{cr,e}$ = the elastic critical shear stress of the web = $\frac{K_v \pi^2 E}{12(1-\mu^2)\left[\frac{d}{t_w}\right]^2}$

μ = Poisson's ratio, and

K_v = 5.35 when transverse stiffeners are provided only at supports

$$= 4.0 + \frac{5.35}{\left(\frac{c}{d}\right)^2}\ for \frac{c}{d} < 1.0$$

$$= 5.35 + \frac{4.0}{\left(\frac{c}{d}\right)^2}\ for \frac{c}{d} \ge 1.0$$

where c, d is the spacing of transverse stiffeners and depth of the web, respectively.

b) Tension field method: The tension field method, which relies on the post-shear buckling strength, can be applied to webs equipped with intermediate transverse stiffeners, as well as those with transverse stiffeners at the supports. This is permissible provided that the panels adjacent to the panel experiencing tension field action, or the end posts, offer adequate anchorage for the tension fields. Additionally, the ratio of the spacing of transverse stiffeners (c) to the depth of the web (d) must not exceed 1.

Nominal shear resistance, V_n, is given by:

$$V_n = V_{tf}$$

where, $V_{tf} = [A_V \tau_b + 0.9 w_{tf} t_w f_V sin\phi] \le V_p$

f_v = yield strength of the tension field obtained from
$= \left[f_{yw}^2 - 3\tau_b^2 + \psi^2\right]^{0.5} - \psi$
$\psi = 1.5\tau_b sin2\phi$

ϕ = inclination of the tension field nearly= $\tan^{-1}\left(\frac{\frac{d}{c}}{1.5}\right)$

w_{tf} = the width of the tension field, given by: $= d\cos\phi - (c - s_c - s_t)\sin\phi$

f_{yw} = yield stress of the web; d = depth of the web

c = spacing of transverse stiffeners in the web;

τ_b= shear stress corresponding to buckling of web

s_c, s_t= anchorage lengths of tension field along the compression and tension flange respectively, obtained from: $s = \frac{2}{\sin\phi}\left[\frac{M_{fr}}{f_{yw}t_W}\right]^{0.5} \leq c$

M_{tf} =reduced plastic moment capacity of the respective flange plate (disregarding any edge stiffener) after accounting for the axial force, N_f in the flange, due to overall bending and any external axial force in the cross-section, and is calculated as:

$$M_{ft} = 0.25 b_f t_f^2 f_{yf}\left[1 - \left(\frac{N_f}{b_f t_f f_{yt}\phi_{mo}}\right)^2\right]$$

b_f, t_f= width and thickness of the relevant flange respectively;

f_{yf}= yield stress of the flange

8.5 Stiffened Web Panels

8.5.1 End Panels Design

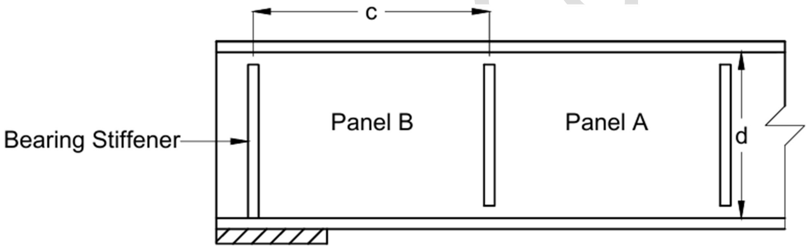


Figure 15: End Panel designed not using Tension Field Action

Notes:

1. Panel A is designed utilizing tension field action

2. Panel B is designed using simple post critical method

3. Bearing Stiffener is designed for the compressive force due to bearing plus compressive force due to the moment M_{tf}

The design of end panels in which interior panel (A as shown in Figure 15) is designed using tension-field method shall be designed according to simplified method.

Moreover, the end panel along with the stiffeners must be evaluated as a beam spanning between the flanges to withstand a shear force, R_f, and a moment, M_f, resulting from the tension field forces. Additionally, the end stiffener shall be capable of withstanding the reaction force plus a compressive force due to the moment, which is equal to M_j.

8.5.2 End Panels Designed using Tension Field Action

The design of end panels in girders, which utilize tension field action, must be conducted in compliance with the provisions specified here. Specifically, the end panel (referred to as Panel B) shall be in compliance with simplified method. Additionally, it shall include an end post made up of either a single or double stiffener (as illustrated in and Figure 15), meeting the following criteria:

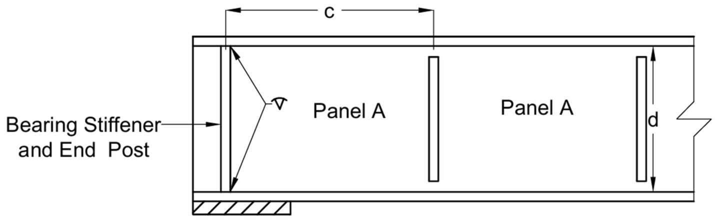


Figure 16: End Panel designed using Tension Field Action (Single Stiffener)

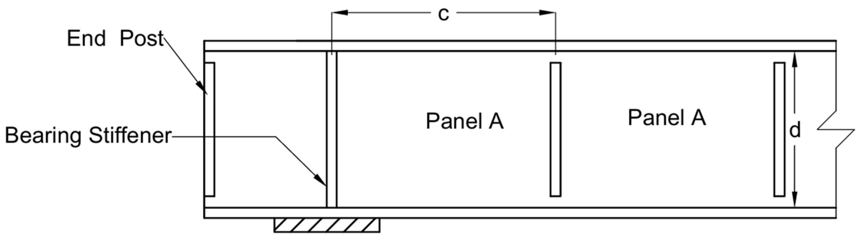


Figure 17: End Panel designed using Tension Field Action (Double Stiffener)

a) Single stiffener (see Figure 16):
   1. The top of the end post shall be rigidly connected to the flange using full-strength welds.
   2. The end post must be capable of withstanding the reaction force plus a moment from the anchor forces equal to $\frac{2}{3} M_f$.
   3. The width and thickness of the end post shall not exceed the width and thickness of the flange.

b) Double stiffener (see Figure 17):
   1. The end post shall be evaluated as a beam spanning between the flanges of the girder.
   2. It must be capable of resisting a shear force R_f and a moment M_f due to the tension field forces.

8.5.3 Anchor Forces

The resulting longitudinal shear, R_{tf}, and a moment M_{tf} from the anchor of tension field forces are calculated as follows: $R_{tf} = \frac{H_q}{2}$ and $M_{tf} = \frac{H_q d}{10}$

where, $H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$; $V_p = \frac{d t f_y}{\sqrt{3}}$; $d =$ web depth

If the actual factored shear force, V in the panel designed using tension field approach is less than the shear strength, V_{tf}, then the values of H_q may be reduced by the ratio $\frac{V\_V_{cr}}{V_{tf} - V_{cr}}$

where, V_{tf} = the basic shear strength for the panel utilizing tension field action
V_{cr} = critical shear strength for the panel designed utilizing tension field action

8.5.4 Panels with Openings: Panels with opening of dimension greater than 10 percent of the minimum panel dimension shall be designed without using tension field action. The adjacent panels shall be designed as an end panel.

8.6 Design of Beams and Plate Girders with Solid Webs

8.6.1 Minimum web thickness: The thickness of the web shall satisfy the following requirements:

8.6.1.1 Serviceability requirements

a) When transverse stiffeners are not provided,

i) $\frac{d}{t_w} \leq 200\epsilon_w$ (Web connected to flanges along both longitudinal edges)

ii) $\frac{d}{t_w} \leq 90\epsilon_w$ (Web connected to flanges along one longitudinal edge only),

b) When only transverse stiffeners are provided (in webs connected to flanges along both longitudinal edges),

i) When $3d \geq c \geq d$

$$\frac{d}{t_w} \leq 200\epsilon_w$$

ii) When $0.74d \leq c < d$

$$\frac{c}{t_w} \leq 200\epsilon_w$$

iii) When $c < 0.74\, d$

$$\frac{d}{t_w} \leq 270\epsilon_w$$

iv) When $c > 3d$, the web shall be considered as unstiffened.

c) When transverse stiffeners and longitudinal stiffeners at one level only are provided (0.2d from compression flange)

i) When $2.4d \geq c \geq d$

$$\frac{d}{t_W} \leq 250\epsilon_w$$

ii) When $0.74d \leq c \leq d$

$$\frac{c}{t_w} \leq 250\epsilon_2$$

iii) When c < $0.74d$

$$\frac{d}{t} t_w \leq 340\epsilon_w$$

d) When a second longitudinal stiffener (located at neutral axis is provided)

$$\frac{d}{t_w} \leq 400\epsilon_w$$

where, d= depth of the web; t_w= thickness of the web;

c = spacing of transverse stiffener; ϵ = yield stress ratio of the web= $\sqrt{\frac{230}{f_{yf}}}$;

f_{yw} = yield stress of the web

8.6.1.2 Compression flange buckling requirements

In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

a) When the transverse stiffeners are not provided: $\frac{d}{t_w} \leq 345\epsilon_f^2$

b) When the transverse stiffeners are provided and

i) When $c \geq 1.5d$

$$\frac{d}{t_w} \leq 345\epsilon_f^2$$

ii) When $c < 1.5d$

$$\frac{d}{t_w} \leq 345\epsilon_f$$

where, d = depth of the web; t_w = thickness of the web;

c = spacing of transverse stiffener; ϵ_f = yield stress ratio of the flange= $\sqrt{\frac{230}{f_{yf}}}$;

f_{yf} = yield stress of compression flange.

8.6.2 Sectional Properties

The effective sectional area of compression flanges shall be the gross area with deductions for excessive width of plates as specified for compression members (see section 7) and for open holes occurring in a plane perpendicular to the direction of stress at the section being considered (see 8.4.1.1).

The effective sectional area of tension flanges shall be the gross sectional area with deductions for holes as specified in 8.4.1.1.

The effective sectional area for parts in shear shall be taken as specified in 8.4.1.1.

8.6.3 Flanges

In bolted construction, flange angles shall comprise as much of the flange area as feasible, ideally not less than one-third. The number of flange plates shall be minimized.

For exposed situations where flange angles are used, at least one plate of the top flange must extend the entire length of the girder, unless the top edge of the web is machined flush with the flange angles.

Each flange plate must extend beyond its theoretical cut-off point and include enough bolts, or welds to develop the load calculated for the bending moment on the girder section (including the curtailed plate) at the theoretical cut-off point.

The projection of flange plates beyond the outer line of connections to flange angles, channel or joist flanges, or in welded constructions, beyond the face of the web or tongue plate, must not exceed local buckling width-to-thickness ratio limits.

8.6.3.1 Flange splices

Flange splices should preferably, not be located at points of maximum stress. Where splice plates are used, their area shall be not less than 5 percent in excess of the area of the flange element spliced; their centre of gravity shall coincide, as nearly as possible, with that of the element spliced. There shall be enough bolts or welds on each side of the splice to develop the load in the element spliced plus 5 percent but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced. In welded construction, flange plates shall be joined by complete penetration butt welds, wherever possible. These butt welds shall develop the full strength of the plates.

8.6.3.2 Connection of flanges to web

The flanges of plate girders must be connected to the web using enough bolts, or welds to transmit the maximum horizontal shear force resulting from the bending moment gradient in the girder, along with any vertical loads directly applied to the flange. If the web is designed using the tension field method, the welds must be capable of transferring the tension field stress, f_{yw}, acting on the web.

8.6.3.3 Bolted constructions

For girders in exposed situations that do not have flange plates extending the entire length, the top edge of the web plate must be flush with or above the angles, while the bottom edge of the web plate must be flush with or set back from the angles.

8.6.3.4 Welded constructions

The gap between the web plates and flange plates shall be kept to a minimum and for fillet welds shall not exceed 1 mm at any point before welding.

8.6.4 Webs

8.6.4.1 Effective sectional area of web of plate girder

The effective cross-sectional area shall be taken as the full depth of the web plate multiplied by the thickness.

Note—Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like, or where the proportion of the web included in the flange area is 25 percent or more of the overall depth, the above approximation is not permissible and the maximum shear stress shall be computed on theory.

8.6.4.2 Splices in webs

Splices and cutouts for service ducts in the webs shall preferably not be located at points of maximum shear force and heavy concentrated loads.

Splices in the webs of plate girders and rolled sections must be designed to resist the shears and moments at the spliced section (See Annex H).

In bolted construction, splice plates shall be provided on each side of the web. In welded construction, web splices shall preferably be made using complete penetration butt welds.

8.6.4.3 When additional plates are needed to enhance the strength of the web, they shall be placed on each side of the web and must be of equal thickness. The portion of the shear force considered to be resisted by these plates shall be limited by the horizontal shear they can transmit to the flanges through their fastenings. These reinforcing plates and their fastenings shall extend up to the points where the flange, without the additional plates, is sufficient.

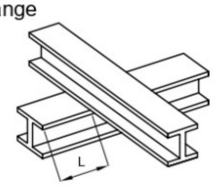
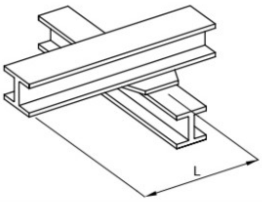
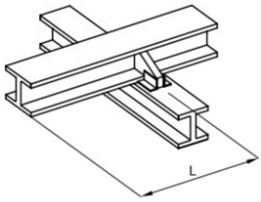
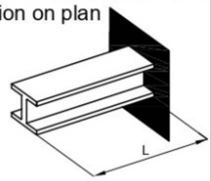
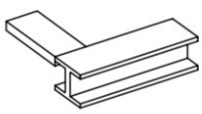
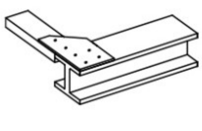
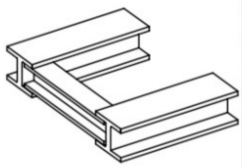
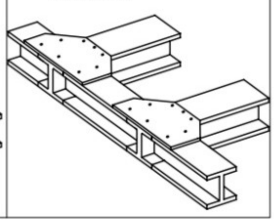
Restraint Condition		Loading Condition	
At Support	At Top	Normal	Destabilizing
(1)	(2)	(3)	(4)
a. Continuous, with lateral restraint to top flange 	1. Free 2. Lateral restraint to top flange 3. Torsional restraint 4. Lateral and torsional restraint	1. 3.0 L 2. 2.7 L 3. 2.4 L 4. 2.1 L	1. 7.5 L 2. 7.5 L 3. 4.5 L 4. 3.6 L
b. Continuous, with partial torsional restraint 	1. Free 2. Lateral restraint to top flange 3. Torsional restraint 4. Lateral and torsional restraint	1. 2.0 L 2. 1.8 L 3. 1.6 L 4. 1.4 L	1. 5.0 L 2. 5.0 L 3. 3.0 L 4. 2.4 L
c. Continuous, with lateral and torsional restraint 	1. Free 2. Lateral restraint to top flange 3. Torsional restraint 4. Lateral and torsional restraint	1. 1.0 L 2. 0.9 L 3. 0.8 L 4. 0.7 L	1. 2.5 L 2. 2.5 L 3. 1.5 L 4. 1.2 L
d. Restrained laterally, torsionally and against rotation on plan 	1. Free 2. Lateral restraint to top flange 3. Torsional restraint 4. Lateral and torsional restraint	1. 0.8 L 2. 0.7 L 3. 0.6 L 4. 0.5 L	1. 1.4 L 2. 1.4 L 3. 0.6 L 4. 0.5 L
Top restraint conditions			
i. Free 	ii. Lateral restraint to top flange 	iii. Torsional restraint 	iv. Lateral and torsional restraint 

Figure 18: Effective Length L_{LT} for Cantilever beam of Length L

8.7 Stiffener Design

8.7.1 General

Web stiffeners shall be employed to provide support to the web when the web is subjected to loads and reactions. They are listed as below:

a) Bearing stiffener: To prevent crushing of web due to concentrated loading.
b) Load carrying stiffener: To resist local buckling of web due to concentrated loading.
c) Tension stiffener: To transfer tensile forces applied to a web through a flange.
d) Intermediate transverse web stiffener: To improve buckling strength of a slender web due to shear.
e) Diagonal stiffeners: To provide local reinforcement to a web in shear and bearing.
f) Torsion stiffeners: To provide torsional restraints at supports.

A stiffener may be designed to perform multiple functions as listed above.

8.7.1.1 Outstand of web stiffeners

Unless the outer edge of a web stiffener is continuously stiffened, the outstand from the face of the web shall not exceed $20t_q\epsilon$.

If the outstand of a stiffener is between $14\epsilon t_q$ and $20\epsilon t_q$, then the stiffener design shall be based on an effective cross-section with an outstand of $14\epsilon t_q$, where t_q is the thickness of the stiffener.

8.7.1.2 Stiff bearing length

The stiff bearing length of any element, b_1, is that length which cannot deform appreciably in bending. To determine b1, the dispersion of load through a steel bearing element should be taken as $45°$ through solid material, such as bearing plates, flange plates, etc (see Figure 19).

8.7.1.3 Eccentricity

In instances where a load or reaction is applied off-center from the web's centerline, or where the centroid of the stiffener is not aligned with the web's centerline, the resultant eccentricity of the loading must be considered in the design process.

8.7.1.4 Buckling resistance of stiffeners

The buckling resistance F_{qd} shall be based on the design compressive stress f_{cd} (see 7.1.2.1) of a strut (curve c), the radius of gyration being taken about the axis parallel to the web. The effective section is the full area or core area of the stiffener (see 8.7.1.2) together with an effective length of web on each side of the centerline of the stiffeners, limited to 20 times the web thickness. The design strength used shall be the minimum value obtained for buckling about the web or the stiffener.

The effective length for intermediate transverse stiffeners used in calculating the buckling resistance, F_{qd}, shall be taken as 0.7 times the length, L of the stiffener.

The effective length for load carrying web stiffeners used in calculating buckling resistance shall be taken as:

a) $KL = 0.7L$ when flange is restrained against rotation inn the plane of the stiffener (by other structural elements),
b) $KL = L$ when flange is not restrained:
where, L = length of the stiffener

When a load or reaction is applied to the flange by a compression member, unless there is effective lateral restraint provided at that point, the stiffener shall be designed as an integral part of the compression member that applies the load. Moreover, the connection between the column and the beam flange must be inspected for the effects of the strut action.

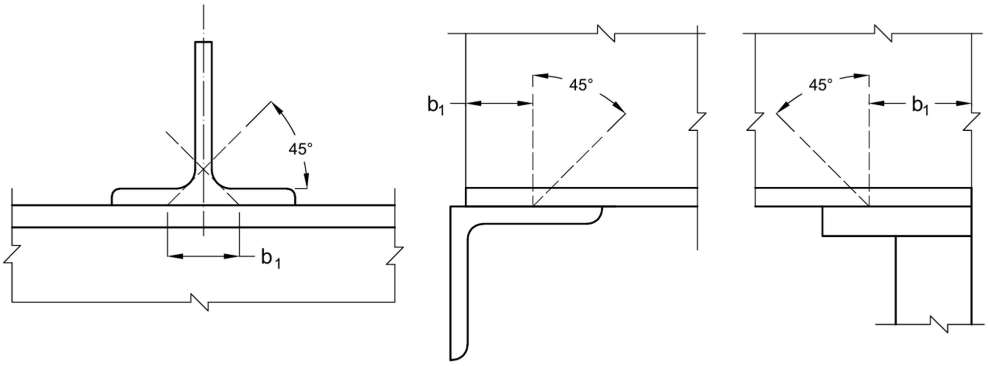


Figure 19: Stiff Bearing Length, b_1

8.7.2 Design of Intermediate Transverse Web Stiffeners

8.7.2.1 Intermediate transverse stiffeners may be provided on one or both sides of the web.

8.7.2.2 Spacing: The spacing of the intermediate stiffeners shall comply with the serviceability requirements of web.

8.7.2.3 Outstand of stiffeners: The outstand of the stiffeners shall comply with provisions mentioned in stiffener design.

8.7.2.4 Minimum stiffeners: Transverse web stiffeners that are not subjected to external loads or moments shall possess a second moment of area, I_s, about the centerline of the web if stiffeners are present on both sides of the web. Alternatively, if a single stiffener is utilized on only one side of the web, the second moment of area shall be about the face of the web. This ensures:

If $\frac{c}{d} \geq \sqrt{2}, \quad I_s \geq 0.75 d t_w^3,$ and if $\frac{c}{d} < \sqrt{2}, \quad I_s \geq \frac{1.5 d^3 t_w^3}{c^2}$

where, d = depth of the web; c= actual stiffener spacing;
t_w= minimum required web thickness for spacing using tension field action.
$\epsilon =$ actual stiffener spacing

8.7.2.5 Buckling check on intermediate transverse web stiffeners

Stiffeners not subjected to external loads or moments shall be checked for a stiffener force:

$$F_q = V - V_{cr}/\gamma_{m0} \leq F_{qd}$$

where, F_{qd}= design resistance of intermediate stiffeners;
$V =$ factored shear force adjacent to the stiffener; and
$V_{cr} =$ shear buckling resistance of the web panel designed without tension field action

Stiffeners subjected to external loads and moments shall meet the conditions for load carrying web stiffeners. They shall in addition satisfy the following interaction equation:

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1$$

where, $F_q =$ stiffener force given above;

$F_{qd} =$ design resistance of an intermediate web stiffener corresponding to buckling about an axis parallel to the web;

$F_x =$ external load or reaction at the stiffener

$F_{xd} =$ design resistance of a load carrying stiffener corresponding to buckling about an axis parallel to the web;

M_q = moment on the stiffener due to eccentrically applied load and transverse load, if any; and

M_{yq} = yield moment capacity of the stiffener based on its elastic modulus about its centroidal axis parallel to the web.

If $F_q < F_x$, then $(F_q - F_x)$shall be taken as zero.

8.7.2.6 Connection of intermediate stiffeners to web

Intermediate transverse stiffeners not subjected to external loading shall be connected to the web so as to withstand a shear between each component of the stiffener and the web (in KN/m) of not less than: $\frac{t_w^2}{5b_s}$

where, t_W= web thickness, in mm ; b_s= outstand width of the stiffener, in mm

For stiffeners subject to external loading, the shear between the web and the stiffener due to such loading has to be added to the above value.

Stiffeners not subject to external loads or moments may terminate clear of the tension flange and in such a situation the distance cut short from the line of the weld should not be more than $4t_w$.

8.7.3 Load Carrying Stiffeners

8.7.3.1 Web check

Load-carrying web stiffeners shall be installed when compressive forces applied through a flange by loads or reactions surpass the buckling strength,F_{cdw}, of the unstiffened web. This strength is computed using the following formula:

The effective length of the web for evaluating the slenderness ratio is calculated as per 9.8.1.4. The area of the cross-section is taken as $(b_1 + n_1)t_w$:

where, b_1= width of stiff bearing on the flange

n_1= dispersion of the load through the web at 45 degrees, to the level of half the depth of the cross-section

8.7.4 Bearing Stiffeners

Bearing stiffeners shall be provided for webs where force applied through a flange by loads or reactions exceeding the local capacity of the web at its connection to the flange, F_w, given by:

$$F_w = (b_1 + n_2)t_w f_{yw}/\gamma_{m0}$$

where, b_1 = stiff bearing length

n_2 = length obtained b dispersion through the flange to the web junction at a slope of 1:25 to the plane of the flange

t_w= thickness of the web; f_{yw}= yield stress of the web

8.7.5 Design of Load Carrying Stiffeners

8.7.5.1 Buckling check

The external load or reaction, F_x on a stiffener shall not exceed the buckling resistance, $F_{sd}$of the stiffener

Where the stiffener also acts as an intermediate stiffener it shall be checked for the effect of combined loads

8.7.5.2 Bearing check

Load carrying web stiffeners shall also be of sufficient size that the bearing strength of the stiffener, F_{psd} given below is not less than the load transferred, F_x

$$F_{psd} = \frac{A_q f_{yq}}{0.8\gamma_{m0}} \geq F_x$$

where, F_x = external load or reaction

A_q = area of the stiffener in contact with the flange; f_{yq} = yield stress of the stiffener

8.7.6 Design of Bearing Stiffeners

Bearing stiffeners should be designed for the applied load or reaction less the local capacity of the web as given in 8.7.4. Where the web and the stiffener material are of different strengths the lesser value should be assumed to calculate the capacity of the web and the stiffener. Bearing stiffeners should project nearly as much as the overhang of the flange through which load is transferred.

8.7.7 Design of Diagonal Stiffeners

Diagonal stiffeners should be designed to carry the portion of the applied shear and bearing that exceeds the capacity of the web.

Where the stiffener also acts as an intermediate stiffener it should be checked for the effect of combined loads in accordance with 8.7.2.5.

8.7.8 Design of Tension stiffeners

Tension stiffeners should be designed to carry the portion of the applied load or reaction less the capacity of the web as given in 8.7.4 for bearing stiffeners.

Where the web and the stiffener are of different strengths, the value for design should be taken as given in 8.7.6.

8.7.9 Torsional Stiffeners

Where bearing stiffeners are required to provide torsional restraint at the supports of the beam, they should meet the following criteria:

a) Capacity calculated as per bearing stiffeners expression as in 8.7.4, and

b) Second moment of area of the stiffener section about the centerline of the web, I_s ,shall be:

$$I_S \geq 0.34\, \alpha_s D^3 T_{cf}$$

where, $\alpha_S = 0.006$ for $\frac{L_{LT}}{r_y} \leq 50;$

$= \frac{0.3}{\left(\frac{L_{LT}}{r_y}\right)}$ for $50 < \left(\frac{L_{LT}}{r_y}\right) \leq 100;$

$= \frac{30}{\left(\frac{L_{LT}}{r_y}\right)^2}$ for $\left(\frac{L_{LT}}{r_y}\right) > 100$

D = overall depth of beam at support,

T_{cf} = maximum thickness of compression flange in the span under consideration

KL = laterally unsupported effective length of the compression flange of the beam

r_y = radius of gyration of the beam about the minor axis

8.7.10 Connection to Web of Load Carrying and Bearing Stiffeners

Stiffeners that resist loads or reactions applied through a flange shall be connected to the web with sufficient welds or fasteners to transmit a design force equal to the lesser of the following:

a) The tension capacity of the stiffener,; and

b) The sum of the forces applied at both ends of the stiffener when they act in the same direction or the larger of the forces when they act in opposite directions.

For stiffeners that do not extend across the entire web, their length shall be such that the shear stress in the web, due to the design force transmitted by the stiffener, does not exceed the web's shear strength. Additionally, the capacity of the web beyond the end of the stiffener shall be adequate to resist the applied force.

8.7.11 Connection to Flanges

8.7.11.1 In tension

Tension-resisting stiffeners shall be connected to the load-transmitting flange using continuous welds or non-slip fasteners.

8.7.11.2 In compression

Compression-resisting stiffeners shall either be fitted against the loaded flange or connected using continuous welds or non-slip fasteners.

The stiffener should be fitted against or connected to both flanges when,

a) a load is applied directly over a support; or
b) it serves as the end stiffener of a stiffened web; or
c) it functions as a torsion stiffener.

8.7.12 Hollow sections

When concentrated loads are applied to hollow sections, local stresses and deformations shall be considered, and the section shall be reinforced as necessary.

8.7.13 Horizontal stiffeners

Where horizontal stiffeners are used in addition to vertical stiffeners, they shall be:

a) When the web thickness is less than the limits specified in section 8.6.1, one horizontal stiffener must be placed on the web at a distance from the compression flange equal to 1/5 of the distance from the compression flange angle, plate, or tongue plate to the neutral axis. This stiffener must be designed such that its I_S , is not less than$4ct_w^3$, where I_S and t_w are defined in section 8.7.2.4, and c is the actual distance between the vertical stiffeners.
b) A second horizontal stiffener, either single or double, shall be placed at the neutral axis of the girder when the web thickness is below the limit specified in section 8.7.1. This stiffener must be designed with a moment of inertia, I_S, not less than $d_2 t_w^2$, where I_S and t_w are defined in section 8.7.2.4, and d_2 is twice the clear distance from the compression flange angles, plates, or tongue plates to the neutral axis.
c) Horizontal web stiffeners shall extend between vertical stiffeners but do not need to be continuous over them. They can be arranged in pairs on each side of the web or as a single stiffener on one side of the web.
d) Horizontal stiffeners may be in pairs arranged on each side of the web, or single located on one side of the web.

8.8 Box Girders

8.8.1 Box girder shall be designed in accordance with specialist literature.

8.8.2 All diaphragms shall be connected so as to transfer the resultant shears to the web and flanges.

8.8.3 Where the concentrated or moving load does not act directly on top of the web, the local effect shall be considered in the design of flanges and the diaphragms.

8.9 Purlins and Sheeting Rails (Girts)

All purlin shall be designed in accordance with the requirements for beams. These limitations of bending stress based on lateral instability of the compression flange shall be considered if the purlin does not receive adequate support from bracings and sheathings in the plane of compression flange. The limitation mentioned for deflection criteria shall be taken in to account. Check shall be made for biaxial bending by calculating bending about two axes (See Section 9).

8.10 Bending in a Non-principal Plane

8.10.1 When the flexural deflection of a member is restricted to a non-principal plane by lateral restraints that prevent lateral deflection, the force exerted by these restraints must be determined. The bending moments about the principal axes acting on the member shall then be calculated from these forces and the applied forces through a rational analysis. The combined effect of bending about the principal axes must meet the requirements of Section 9.

8.10.2 When the deflections of a member loaded in a non-principal plane are unconstrained, the bending moments about the principal axes shall be calculated using a rational analysis. The combined effect of bending about the principal axes must meet the requirements of Section 9.

8.11 Restraints

8.11.1 Intermediate lateral restraints

If a member that is subject to bending needs intermediate lateral restraints within its length in order to develop the required buckling resistance moment, these restraints shall have sufficient stiffness and strength to inhibit lateral movement of the compression flange relative to the supports. The intermediate lateral restraints shall be either connected to a system capable of effective force transfer.

Intermediate lateral restraints shall generally be connected to the member as close as practicable to the compression flange and in any case closer to the level of the shear center of the compression flange than to the level of the shear center of the member. However, if an intermediate torsional restraint is also provided at the same cross-section, an intermediate lateral restraint is allowed to be connected at any level.

8.11.2 Restraint forces

8.11.2.1 When beams are provided with members giving effective lateral restraint at intervals along the span, the effective lateral restraint shall be capable of resisting a force of 2.5 percent of maximum factored force in the compression flange divided equally between the points where restraint members are provided. Furthermore, each restraint point shall be capable of resisting 1 percent of the maximum force in the compression flange.

8.11.2.2 In a series of such beams, with solid webs, which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum flange force in one beam only.

8.11.2.3 In the case of a series of latticed beams, girders or roof trusses which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum force in the compression flange plus 1.25 percent of this force for every member of the series other than the first, up to a maximum total of 7.5 percent.

8.11.2.4 Purlins adequately restrained by sheeting need not normally be checked for forces caused by restraining rafters of roof trusses or portal frames that carry predominantly roof loads, provided that either:

a) there is bracing of adequate stiffness in the plane of the rafters; or
b) the roof sheeting is capable of acting as a stressed-skin diaphragm

8.11.2.5 In case of beams with double curvature bending, adequate direct lateral support to the compression flange in the hogging moment region maybe provided as given above for simply supported beam. The effect of support to the tension (top) flange in the hogging moment region on lateral restraint to the compression flange may be considered as per specialist literature.

9 Member Subjected to Combined Forces

9.1 General

This standard governs the design of members subjected to combined forces, such as shear force and bending, axial force and bending, or shear force, axial force and bending.

9.2 Combined Shear and Bending

There are two cases possible when the action of both shear and bending are to be considered on a member.

9.2.1 High Shear Case

High Shear case is considered when the shear force on a section is greater than or equal to 50 percent of the shear strength of the section. In this case

a) Plastic and compact sections: $M_c = p_y\left(Z_p - \beta\, Z_v\right) \leq\ 1.2\, Z_e p_y$
b) Semi-compact sections: $M_c = p_y Z_e$

where, $p_y =$ design strength of the material i.e. p_y=f_y/γ_{mo}
$Z_e =$ elastic section moduli of the cross-section;
$Z_p =$ plastic section moduli of the cross-section
$Z_v = Z_p - Z_f$, where Z_f is the plastic section modulus of the area excluding the shear area
$\beta = \left(2\frac{V}{V_d} - 1\right)^2$, where V is factored shear force and V_d is shear strength as governed by web yielding or web buckling.

9.2.2 Low Shear Case: In this case, no reduction is necessary in the bending capacity of the member.

9.3 Combined Axial Force and Bending Moment

In case of section exposed to both axial force and bending moment, two checks are needed:

a) Section strength as governed by Material Failure check
b) Member strength as governed by Buckling Failure check

9.3.1 Section strength

9.3.1.1 Plastic and compact sections

For designing members subjected to axial force, either tension or compression and bending moment, the following should be satisfied:

$$\left(\frac{M_y}{M_{ndy}}\right)^2 + \left(\frac{M_z}{M_{ndz}}\right)^2 \leq 1$$

Conservatively, the following equation may also be used under combined axial force and bending moment:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1$$

where, M_y, M_z = factored applied moments about minor and major axis of cross-section

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone
N=factored applied axial force (Tension, T or Compression, P)
N_d= design strength in tension, T_d or in compression due to yielding given by $N_d = A_g f_y/\gamma_{m0}$
M_{dy}, M_{dz} =design strength under corresponding moment acting alone
A_g= gross area of the cross-section
α_1, α_2=constants (Table 19); γ_{m0} = partial factor of safety in yielding

9.3.1.2 For plastic and compact sections without bolts holes, the following approximations may be used for evaluating M_{ndy} and M_{ndz}:

a) Plates

$$M_{nd} = M_d(1 - n^2)$$

b) Welded I or H sections

$$M_{ndy} = M_{dy}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \leq M_{dy}\ where\ n \geq a$$

$$M_{ndz} = \frac{M_{dz}(1-n)}{1-0.5a} \leq M_{dz}$$

where,

$$n = N/N_d \text{ and } \alpha = \frac{A - 2bt_f}{A} \leq 0.5$$

c) For standard I or H sections

$$M_{nfz} = 1.11\ M_{dz}(1-n) \leq M_{dz}$$

For $n \leq 0.2$, $M_{ndy} = M_{dy}$

For $n > 0.2$, $M_{ndy} = 1.56 M_{dy}(1-n)(n+0.6)$

$M_{ndz} = 1.11 M_{dz}(1-n) \leq M_{dz}$

d) For rectangular hollow sections and welded box sections

When the section is symmetric about both axes and without bolt holes

$$M_{ndy} = \frac{M_{dy}(1-n)}{1-0.5a_f} \leq M_{dy}$$

$$M_{ndz} = \frac{M_{dz}(1-n)}{1-0.5a_w} \leq M_{dz}$$

where,

$$a_w = \frac{A - 2bt_t}{A} \leq 0.5$$

$$a_f = \frac{A - 2ht_w}{A} \leq 0.5$$

e) Circular hollow tubes without bolt holes

$$M_{nd} = 1.04 M_d(1 - n^{1.7}) \leq M_d$$

9.3.1.3 Semi-compact section

In the absence of high shear force, semi-compact section design is satisfactory under combined axial force and bending, if the maximum longitudinal stress under combined axial force and bending, f_x satisfies the following criteria:

$$f_x \leq f_y/\gamma_{mo}$$

For cross-section without holes, the above criteria reduces to,

$$\frac{N}{N_d} + \frac{M}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1$$

where, $N_d, M_{dy}\ and\ M_{dz}$ are as defined 9.3.1.1.

Table 19: Constants α_1 and α_2

SN	Section	α_1	α_2
1	I and Channel	$5n \geq 1$	2
2	Circular tubes	2	2
3	Rectangular tubes	$1.66/(1 - 1.13n^2) \leq 6$	$1.66/(1 - 1.13n^2) \leq 6$
4	Solid rectangles	$1.73 + 1.8n^3$	$1.73 + 1.8n^3$

Note: n = N/N_d

9.3.2 Overall Member Strength

Members subjected to combined axial force and bending moment shall be checked for overall buckling failure as given in this section.

9.3.2.1 Bending and axial tension

The reduced effective moment, M_{eff}, under tension and bending calculated as given below, should not exceed the bending strength due to lateral torsional buckling, M_d.

$$M_{eff} = \left[M - \frac{\psi T Z_{ec}}{A} \right] \leq M_d$$

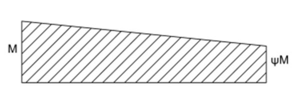
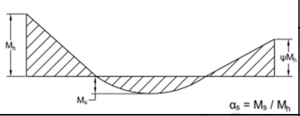
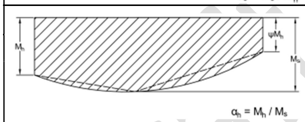
where, $M, T =$ factored applied moment and tension, respectively
$A =$ area of cross-section
Z_{ec} =elastic section modulus of the section with respect to extreme compression member
$\psi = 0.8$, if T and M can vary independently,or otherwise$= 1.0$

9.3.2.2 Bending and axial compression

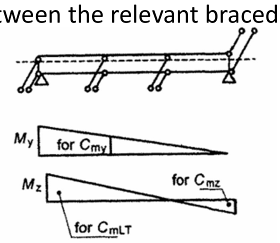
The reduced effective moment, M_{eff}, under tension and bending calculated as given below, should not exceed the bending strength due to lateral torsional buckling, M_d.

$$M_{eff} = \left[M - \frac{\psi T Z_{ec}}{A} \right] \leq M_d$$

where, $M, T =$factored applied moment and tension, respectively
$A =$ area of cross-section
Z_{ec} =elastic section modulus of the section with respect to extreme compression member
$\psi = 0.8$, if T and M can vary independently, or otherwise $= 1.0$

Bending Moment Diagram	**Range**		**C_{my}, C_{mz}, C_{mLT}** Uniform Loading	**C_{my}, C_{mz}, C_{mLT}** Concentrated Load
	$-1 \leq \psi \leq 1$		$0.6 + 0.4\psi \geq 0.4$	
	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0.2 + 0.8\alpha_s \geq 0.4$	$0.2 + 0.8\alpha s \geq 0.4$
	$-1 \leq \alpha s \leq 0$	$0 \leq \psi \leq 1$	$0.1 - 0.8\alpha s \geq 0.4$	$-0.8\alpha s \geq 0.4$
		$-1 \leq \psi \leq 0$	$0.1(1-\psi) - 0.8\alpha s \geq 0.4$	$0.2(1-\psi) - 0.8\alpha s \geq 0.4$
	$0 \leq \alpha h \leq 1$	$-1 \leq \psi \leq 1$	$0.95 - 0.05\alpha h$	$0.90 + 0.10\alpha h$
	$-1 \leq \alpha h \leq 0$	$0 \leq \psi \leq 1$	$0.95 + 0.05\alpha h$	$0.90 + 0.10\alpha h$
		$-1 \leq \psi \leq 0$	$0.95 + 0.05\alpha h(1 + 2\psi)$	$0.90 + 0.1\alpha h(1 + 2\psi)$
For members with sway buckling mode, the equivalent uniform moment factor $C_{my} = C_{mz} = 0.9$.				
C_{my}, C_{mz}, C_{mLT} shall be obtained according to the bending moment diagram between the relevant braced points				

Moment factor	**Bending axis**	**Points braced in direction**
C_{my}	$z - z$	$y - y$
C_{mz}	$y - y$	$z - z$
C_{mLT}	$z - z$	$z - z$



10 Connections

10.1 General

10.1.1 This section covers the requirements for the design and detailing of the connection of steel members. Connection elements consist of cleats, gussets, connectors, connecting plates, brackets, and connectors which consist of bolts, pins, and welds

10.1.2 A connection should be designed to transmit the necessary design forces in members along with additional design effects. However, ease of fabrication and erection should be considered in the design of connections. Special attention shall be paid to check site hole clearances, tightening of fasteners, and welding procedures.

10.1.3 Effects of residual stresses in addition to stress due to tightening of fasteners and normal tolerances of fit-up may not be considered in connection design, provided that ductile behavior is ensured.

10.1.4 In general, the use of different forms of fasteners to transfer the same force shall be avoided. However, when different forms of fasteners are used to carry a shear load or when welding and fasteners are combined, then one form of fastener shall be normally designed to carry the total load. Nevertheless, fully tensioned friction grip bolts may be designed to share the load with welding, provided the bolts are fully tightened to develop the necessary pretension after welding.

10.1.5 The partial safety factor in the evaluation of design strength of connections shall be taken as given in Table 4.

10.2 Bolt Hole Details

10.2.1 Clearance for bolt holes

Three types of holes may be provided:

a) Standard clearance hole: These are normal hole clearances used for a given diameter of a bolt. The values for hole clearances are given in Table 20.
b) Oversize hole: These are holes greater than standard clearance holes and can be used in slip-resistant connections and hold-down bolted connections. However, these should be covered by a cover plate of sufficiently large size and thickness and have a hole not larger than the standard clearance hole. The values for hole clearances are given in Table 20.
c) Short and long slots: Short and long slot holes are larger than standard clearance holes and are also used in slip-resistant connections and hold-down bolted connections. These should also be covered by a cover plate of sufficiently large size and thickness and have a hole not larger than the standard clearance hole. The values for hole clearances are given in Table 20.

Table 20: Clearance for Bolt Holes

SN	Nominal Size of the bolt, d, (mm)	Size of Hole = Nominal diameter of Bolt + Clearances			
		Standard Clearance in Diameter (mm)	Oversize Clearance in diameter (mm)	Clearance in Length of Short Slot (mm)	Clearance in Length of Long Slot (mm)
i)	12-14	1.0	3.0	4.0	$2.5d$
ii)	16-22	2.0	4.0	6.0	$2.5d$
iii)	24	2.0	6.0	8.0	$2.5d$
iv)	Larger than 24	3.0	8.0	10.0	$2.5d$

10.3 Spacing Requirements

10.3.1 Pitch Requirement

The minimum distance between any two bolt holes in the direction of the stress should be less than 3d (d is diameter of bolt).

The maximum distance between any two bolt holes in the direction of the stress should not be less than 12t, where, t is thickness of connected element but $< 150\ mm$.

10.3.2 Edge and End Distances

The minimum edge distances from center of any hole to nearest edge of a plate shall not exceed 1.75 times the hole diameter in case of sheared or hand-flume cut edges and 1.5 times the hole diameter in case of rolled, machine-flame cut, sawn and planed edge.

The maximum edge distance to the nearest line of bolts from an edge or any un-stiffened part should not exceed $11t\epsilon$, where $\epsilon = \left(\frac{230}{f_y}\right)^{\frac{1}{2}}$. Where, the members are exposed to corrosive influences, the maximum edge distances shall not exceed 40mm plus 4t.

10.4 Bearing Type Bolts

10.4.1 Effective Area of Bolts

10.4.1.1 The actual area of the bolt intersecting with the shear plane should be used in calculation. For example, if A_e be the cross-sectional area of the bolts without threads then the cross-sectional area of the bolts if required in calculation should be taken as 78% of A_e.

10.4.2 A bolt subjected to a factored shear force (V_{sb}) shall satisfy the condition:

$$V_{sb} = V_{db}$$

where, V_{db} is the design strength of the bolt taken as the smaller of the value as governed by shear, V_{dsb} (see 10.4.3)and bearing, V_{dpb} (see 10.4.3).

10.4.3 Shear Capacity of Bolt

The design strength of the bolt, V_{dsb} as governed shear strength is given by:

$$V_{dsb} = V_{nsb}/\gamma_{mb}$$

where, V_{nsb} =nominal shear capacity of a bolt, calculated as: $V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$

where, f_u = ultimate tensile strength of a bolt;
n_n = number of shear planes with threads; n_s = number of shear planes without threads;
A_{sb} = nominal plain shank area of the bolt; A_{nb} = net shear area of the bolt at threads.

10.4.3.1.1 Modification factors for shear capacity of bolt

a. Long Joints (β_{lj})
When the length of the joint, l_j of a splice or end connection in compression or tension element containing more than two bolts exceeds $15d$ in the direction of the load, the nominal shear capacity, V_{db} shall be reduced by the factor β_{lj}, given by:

$$\beta_{lj} = 1.075 - 0.005\left(\frac{l_j}{d}\right) \text{ but } 0.75 \le \beta_{lj} \le 1$$

Where, d = Nominal diameter of the fastner

NOTE —This provision does not apply when the distribution of shear over the length of joint is uniform, as in the connection of web of a section to the flanges.

b. Large grip lengths (β_{lg})
When the grip length, l_g (equal to total thickness of the connected plates) exceeds 5 times the diameter, d of the bolts, the design shear capacity shall be reduced by a factor β_{lg}, given by:

$$\beta_{lg} = \frac{8d}{(3d + l_g)}$$

β_{lg} shall not be more than β_{lj}. The grip length, l_g shall in no case be greater than 8d.

c. Packing plates

The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by:

$$\beta_{pk} = (1 - 0.0125 t_{pk})$$

where, t_{pk} = thickness of the thicker pacing, in mm.

10.4.4 Bearing Capacity of the Bolt

The design bearing strength of a bolt on any plate, V_{dpb} as governed by bearing is given by:

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

where, V_{npb} = nominal bearing strength of a bolt= $2.5\, k_b d\, t\, f_u$

k_b is the smaller of $\left\{\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1\right\}$

e, p = end and pitch distances of the fastener along bearing direction; d_0 =diameter of the hole;

f_{ub}, f_u = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate

d = nominal diameter of the bolt;

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking.

The bearing resistance (in direction normal to the slots in slotted holes) of bolts in holes other than standard clearance holes may be reduced by multiplying the bearing resistance obtained as above, V_{npb} by the factors given below:

a) Over size and short slotted holes – 0.7, and
b) Long slotted holes – 0.5

10.4.5 Tension Capacity

A bolt subjected to a factored tensile force, T_b shall satisfy: $T_b \le T_{db}$

where, $T_{db} = T_{nb}/\gamma_{mb}$

T_{nb} = nominal tensile capacity of the bolt, calculated as: $0.90 f_{ub} A_n < f_{yb} A_{sb} \left(\frac{\gamma_{mb}}{\gamma_{m0}}\right)$

where, f_{ub} = ultimate tensile stress of the bolt; f_{yb} = yield stress of the bolt;

A_n = net tensile stress area; A_{sb} = shank area of the bolt.

10.4.6 Bolts Subjected to Combined Shear and Tension

A bolt required to resist both design shear force (V_{sd}) and design tensile force (T_b) at the same time shall satisfy:

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

where, V_{sb} =factored shear force acting on the bolt; V_{db} =design shear capacity (See 10.4.2)

T_b =factored tensile force acting on the bolt; T_{db} =design tension capacity (See 10.4.5)

10.5 Friction Grip Type Bolting

10.5.1 In friction grip type bolting, initial pretension in bolt develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping forces opposes slip due to externally applied shear. Friction grip type bolts and nuts shall conform to IS 3757. Their installation procedures shall conform to IS 4000.

10.5.2 Where the slip between bolted plates cannot be tolerated at working loads (slip critical connections), the requirements of 10.5.3 shall be satisfied. However, at ultimate loads, the requirements of 10.5.4 shall be satisfied by all connections.

10.5.3 Slip Resistance

Design for friction type bolting in which slip is required to be limited, a bolt subjected only to a factored design shear force, V_{sf} in the interface of connections at which slip cannot be tolerated, shall satisfy the following:

$$V_{sf} \leq V_{dsf}$$

where, $V_{dsf} = V_{nsf}/\gamma_{mf}$

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as:

$$V_{nsf} = \mu_f n_e K_h F_o$$

where, μ_f = coefficient of friction (slip factor) as specified in Table 21 ($\mu_f \leq 0.55$);

n_e = number of effective interfaces offering frictional resistance to slip;

K_h = 1.0 for fasteners in clearance holes,

= 0.85 for fasteners in oversized and short slotted holes and for fasteners in long slotted holes loaded perpendicular to the slot,

= 0.7 for fasteners in long slotted holes loaded parallel to the slot;

$\gamma_{mf} = 1.10$ (if slip resistance is designed at service load),

= 1.25 (if slip resistance is designed at ultimate load);

F_u = minimum bolt tension (proof load) at installation and may be taken as $A_{nb} f_0$,

A_{nb} =net area of the bolt at threads; f_o = proof stress ($= 0.70 f_{ub}$)

NOTE — V_{nsf} may be evaluated at a service load or ultimate load using appropriate partial safety factors, depending upon whether slip resistance is required at service load or ultimate load.

10.5.3.1 Long Joints

The provision for the long joints of bearing type bolts also apply to friction grip connections.

10.5.4 Capacity after Slipping

When friction type bolts are designed not to slip only under service loads, the design capacity at ultimate load may be calculated as per bearing type connection (see 10.4.2 and 10.4.3).

NOTE —The block shear resistance of the edge distance due to bearing force may be checked as given in 6.4.

10.5.5 Tension Resistance

A friction bolt subjected to a factored tension force (T_f) shall satisfy:

$$T_f \leq T_{df}$$

where, $T_{df} = \frac{T_{nf}}{\gamma_{mf}}$

T_{nf} = nominal tensile strength of the friction bolt, calculated as: $T_{nf} = 0.9\, f_{ub} A_n \leq f_{yb} A_{sb} \left(\frac{\gamma_{m1}}{\gamma_{m0}}\right)$

f_{ub} = ultimate tensile stress of the bolt; A_n = net tensile stress area;

A_{sb} = shank area of the bolt; γ_{mf} = partial factor of safety.

Table 21: Typical Average Values for Coefficient of Friction(μ_f)

SN	Treatment of Surface	Coefficient of Friction, μ_f
i)	Surfaces not treated	0.20
ii)	Surfaces blasted with shot or grit with any loose rust removed, no pitting	0.50
iii)	Surfaces blasted with shot or grit and hot-dip galvanized	0.10
iv)	Surfaces blasted with shot or grit and spray-metalized with zinc (thickness 50–70 µm)	0.25
v)	Surfaces blasted with shot or grit and painted with ethyl zinc silicate coat (thickness 30–60 µm)	0.30
vi)	Sand blasted surface, after light rusting	0.52
vii)	Surfaces blasted with shot or grit and painted with ethyl zinc silicate coat (thickness 60–80 µm)	0.30
viii)	Surfaces blasted with shot or grit and painted with alkalized silicate coat (thickness 60–80 µm)	0.30
ix)	Surface blasted with shot or grit and spray metalized with aluminum (thickness >50 µm)	0.50
x)	Clean mill scale	0.33
xi)	Sand blasted surface	0.48
xii)	Red lead painted surface	0.10

10.5.6 Combined Shear and Tension

Bolts for which slip in serviceability limit state shall be limited, which are subjected to tension force, T, and shear force, V, shall satisfy:

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

where, V_{sf} = applied factored shear force at design load; V_{df} = design shear strength;
T_f = externally applied factored tension at design load; T_{df} = design tension strength.

10.5.7 Prying force

Where prying force, Q as illustrated in Figure 20 shall be calculated as given below: and added to the tension in the bolt:

$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_o b_e t^4}{27 l_e l_v^2}\right]$$

where, l_v = distance from the bolt centerline to the toe of the fillet weld or to half the root radius for a rolled section;
l_e = distance between prying force and bolt centerline and is the minimum of either the end distance or the value given by:

$$l_e = 1.1t\sqrt{\frac{\beta f_o}{f_y}}$$

Where, $\beta =$ 2 for non pre-tensioned bolt and 1 for pre-tensioned bolt; $\eta = 1.5$

b_e = effective width of flange per pair of bolts; f_o = proof stress in consistent units;
t = thickness of the end plate.

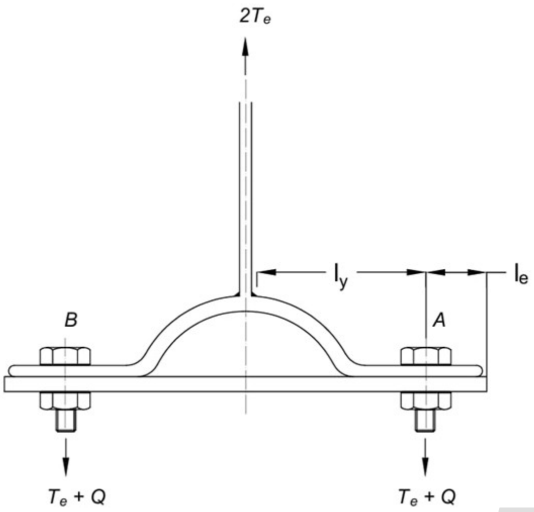


Figure 20: Combined Prying force and tension

10.6 Pin Connections

10.6.1 Shear Capacity

A pin subjected to design shear force (V_f^*) shall satisfy:

$$V_f^* \leq \frac{V_f}{\gamma_{mb}}$$

where, γ_{mb} = partial safety factor
V_f = nominal shear capacity of the pin

The nominal shear capacity of the pin shall be calculated as follows:

$$V_f = \frac{f_u}{\sqrt{3}}(n_s A_p)$$

where, f_u = ultimate stress of the pin material
n_s = number of shear planes
A_p = cross-sectional area of the pin

10.6.2 Bearing Design

A pin and a ply subjected a design bearing force (V_b^*) shall satsify:

$$V_b^* \leq V_b/\gamma_{mb}$$

where, γ_{mb} = partial safety factor
V_b = nominal bearing capacity of the pin or ply, whichever is lesser

The nominal bearing capacity (V_b) shall be calculated as follows:

$$V_b = 1.4 f_y d_f t_p k_p$$

where, f_{yb} = yield stress of the pin or the ply, whichever is lesser
d_f = pin diameter
t_p = connecting plate thickness

$k_p = 1.0$ for pins without rotation, or,
$= 0.5$ for pins with rotation.

10.6.3 Bending Capacity

A pin subjected to a design bending moment (M^*) shall satsify:

$$M^* \leq M_p/\phi$$

where, $\phi =$ partial safety factor
V_b = nominal moment capacity of the pin

The nominal moment capacity of a pin (M_p) shall be calculated as:

$$M_p = f_{yp}S$$

where, $f_{yp} =$ yield stress of the pin
$S =$ plastic section modulus of the pin

The design details relating to minimum and maximum pitch, minimum and maximum edge and end distances shall be as per given for bolts.

10.7 Welds and Welding

10.7.1 General

Requirements of welds and welding shall conform to IS 816 and IS 9595, as appropriate.

10.7.2 End Return or Boxing

Fillet welds shall not be terminated at the extreme ends or edges of members. The end returns shall extend at least twice the weld size (unless it is impracticable to do so), but not more than four times the weld size.

10.7.3 For the purpose of design, welds may be classified as:

a) Fillet welds
   i) Continuous welds
   ii) Intermittent welds
   iii) Plug welds on circular and elongated holes
   iv) Slot welds

b) Butt welds
   i) Full penetration butt welds
   ii) Partial penetration butt welds
   iii) Butt welds reinforced with fillet welds

10.7.4 Specification for Fillet weld

10.7.4.1 Size

The nominal size of a normal fillet weld shall be taken as the minimum weld leg size, taken as a distance from the root to the toe of the fillet weld. For deep penetration welds, where the depth of penetration beyond the root run is a minimum of 2.4 mm, the size of the fillet should be taken as the minimum leg size plus 2.4 mm.

10.7.4.1.1 Maximum size of weld

The maximum size of the fillet weld is a function of the thickness of the thinner plates joined.

a) If the thickness of thinner plate is less than 6 mm, the maximum leg length should be the thickness of the thinner plate.
b) If the thickness of the plate is equal to or greater than 6 mm, the maximum leg length should be the thickness of the plate minus 1.5 mm
c) Where the fillet weld is applied to the rounded toe of a rolled section, the specified size of the weld shall not exceed $\frac{3}{4}$ of the thickness of the section at the toe.

10.7.4.1.2 Minimum size of weld

The size of fillet welds shall not be less than 3 mm. The minimum size of the first run or of a single run fillet weld shall be as given in Table 22, to avoid the risk of cracking in the absence of preheating.

Table 22: Minimum size of a fillet weld

SN	Minimum size of First Run or of a single run fillet weld		Minimum size (mm)
	Over	**Up to and including**	
1	-	10	3
2	10	20	5
3	20	32	6
4	32	50	8 of first run 10 for minimum size of weld

Notes:

1. When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to percent cracking of the weld. Where the thicker part is more than 50 mm thick, special precaution like pre-heating should be taken place.

10.7.4.2 Effective throat thickness

10.7.4.2.1 The effective throat thickness of a fillet weld shall not be less than 3 mm and shall generally not exceed $0.7t$ or $1.0t$ under the special circumstance, where t is the thickness of the thinner plate of elements being welded.

10.7.4.2.2 For the purpose of stress calculation in fillet welds joining faces inclined,

$$\text{Effective throat thickness} = \text{K} \times \text{size of the weld} = \text{KS}$$

where, S is the size of the weld in mm, and K is the constant. The value of K depends on the angle between the fusion faces and is given in Table 23.

Table 23: Values of K for Different Angles Between Fusion Faces

Angle between fusion faces	$60° - 90°$	$91° - 100°$	$101° - 106°$	$107° - 113°$	$114° - 120°$
Constant, K	0.70	0.65	0.60	0.55	0.50

Note: Fillet weld is not recommended if the angle between fusion faces is less than 60° or more than 120°.

10.7.4.3 Length

10.7.4.3.1 Effective Length (Straight)

The effective length of a fillet weld should be taken as the length over which the fillet is full size. In the absence of better information this may be taken as equal to the overall length, less s for each end that does not continue around a corner. In practice the actual length of weld is made of the effective length shown in the drawing plus two times the weld size, but not less than four times the size of the weld.

10.7.4.3.2 Effective Length (Curve)

The effective length of a curve fillet weld shall be measured along the centerline of the effective throat.

10.7.4.3.3 Minimum Length

The minimum effective length of a fillet weld shall be at least four times the nominal size, or the effective size of the weld shall be considered not to exceed 25% of its effective length.

10.7.4.4 Effective Area

The effective area shall be the effective weld length multiplied by the effective throat. Stress in a fillet weld shall be considered as applied to this effective area, for any direction of applied load.

10.7.4.5 Overlap

In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 40 mm.

10.7.4.6 Transverse Spacing

When the end of an element is connected only by longitudinal fillet weld, the length of the weld along either edge should not be less than the transverse spacing between longitudinal welds.

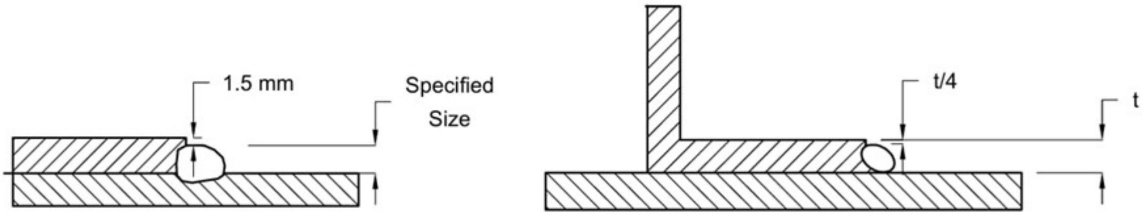


Figure 21: Fillet welds on square edge of plate or rounded toe of rolled section.

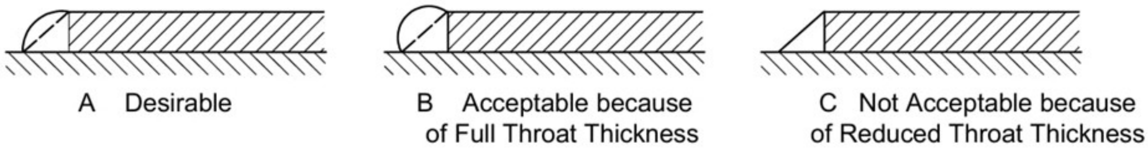


Figure 22: Full size fillet weld applied to the edge of a plate or section.

10.7.4.7 Design Strength of Fillet weld

The design stress of a fillet weld

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

where, f_{wd} = nominal strength of fillet weld = $f_u/\sqrt{3}$

f_u = smaller of the ultimate stress of the weld or of the parent metal

γ_{mw} = Partial safety factor

The design strength of a fillet weld is based on its throat thickness area and is given by

$$P_{dw} = l_w t_t \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

where, t_t = throat thickness of fillet weld = KS; S = Size of fillet weld

10.8 Plug and Slot Welds

10.8.1 Plug or slot welds are permitted to be used to transmit shear, to prevent the buckling or separation of lapped parts, and to interconnect the components of built-up members; however, they should not be used to resist externally applied tension.

10.8.2 Effective Area: The effective area of plug and slot welds shall be taken as the nominal area of the hole or slot in the plane of the faying surface.

10.8.3 The diameter of a circular hole, or width of an elongated hole, for a plug weld should be at least 8 mm more than the thickness of the part containing it.

10.8.4 Minimum Spacing (Plug Welds): The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

10.8.5 Minimum Spacing (Slot Welds): 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.

10.8.6 The center to center spacing of plug welds shall not exceed the value necessary to prevent local buckling

10.8.7 The length of slot for a slot weld shall not exceed 10 times the thickness of the weld.

10.8.8 Slot Ends: 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.

10.8.9 The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the length of the slot

10.8.10 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.

10.9 Long Joints

When the length of the fillet weld exceeds 100 times the weld size, the design stress of weld,f_{wd} shall be reduced by the factor.

$$\beta = 1.2 - 0.2\frac{l_j}{150\, t_t} \leq 1.0$$

where, $L =$ Actual Length of the joint in the direction of the force transfer, mm;

$t_t =$ throat size of the weld, mm

10.9.1 Fillet Weld Applied to the Edge of a Plate or section

10.9.1.1 Where a fillet weld is applied to the square edge of a part, the specified size of the weld should generally be at least 1.5 mm less than the edge thickness in order to avoid washing down of the exposed corner (see Figure 21).

10.9.1.2 Where the fillet weld is applied to the rounded toe of a rolled section, the specified size of the weld should generally not exceed ¾ of the thickness of the section at the toe (see Figure 21).

10.9.1.3 Where the size specified for a fillet weld is such that the parent metal will not project beyond the weld, no melting of the outer cover or covers shall be allowed to occur to such an extent as to reduce the throat thickness (see Figure 22).

10.9.1.4 When fillet welds are applied to the edges of a plate or section in members subject to dynamic loading, the fillet weld shall be of full size with its leg length equal to the thickness of the plate or section, with the limitation specified in 10.9.1.3.

10.9.1.5 End fillet welds, normal to the direction of force shall be of unequal size with a throat thickness not less than $0.5t$, where t is the thickness of the part, as shown in Figure 24. The difference in thickness of the welds shall be negotiated at a uniform slope.

10.10 Specification for Groove/Butt Welded Joints

10.10.1 Effective weld length: The maximum effective weld length for any groove weld, square or skewed, shall be the width of the part jointed, perpendicular to the direction of tensile or compressive stress. For groove welds transmitting shear, the effective length is the length specified.

10.10.2 Size: The size of the groove weld used is specified by the throat dimension.

10.10.2.1 Minimum Size: The minimum effective throat thickness of a fillet weld shall not be less than 3 mm.

10.10.2.2 Maximum Size: The maximum effective throat shall not exceed 0.7t, or 1.0t under special circumstances, where t is the thickness of the thinner plate of elements being welded.

10.10.3 Effective Area: The effective area of the groove welds shall be taken as the length of the weld times the effective throat.

10.10.4 Design Strength of Groove Weld

10.10.4.1 The design strength of the groove weld in tension or compression is given by:

$$T_{dw} = \frac{f_{y1} l_w t_e}{\gamma_{mw}}$$

where, f_{y1} = smaller of yield stress of the weld (f_{yw}) and the parent metal (f_y) in MPa

l_w = effective length of the weld in mm;

t_e = effective throat thickness of the weld in mm; γ_{mw} = partial safety factor

10.10.4.2 The design strength of the groove weld in shear is given by

$$V_{dw} = \frac{f_{yw1}\, l_w t_e}{\gamma_{mw}}$$

where, f_{yw1} = smaller of shear stress of weld ($f_{yw}/\sqrt{3}$) and the parent metal ($f_y/\sqrt{3}$) in MPa

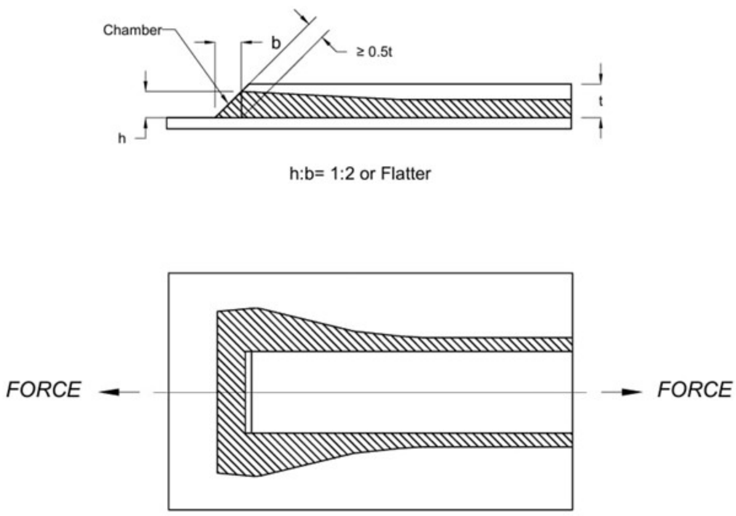


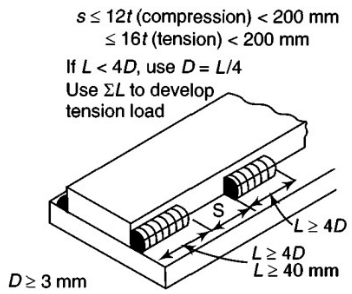
Figure 24: End fillet weld normal to direction of force

10.11 Intermittent Welds

10.11.1 Intermittent butt and fillet welds are permitted to be used to transfer calculated stress across a joint or faying surfaces and to join components of built-up members. The length of any segment of intermittent butt and fillet welding shall be not less than four times the weld size, with a minimum of 40 mm.

10.11.2 The clear spacing between the effective lengths of intermittent butt and fillet weld shall not exceed 12 and 16 times the thickness of thinner plate joined, for compression and tension joint respectively, and in no case be more than 200 mm.

10.11.3 The intermittent welds shall not be used in positions subject to dynamic, repetitive and alternating stresses.



Intermittent weld dimensions

10.12 Built-up members- intermittent fillet welds

10.12.1 If intermittent fillet welds connect components forming a built-up member, the welds shall comply with the following requirements:

a) At the ends of a tension or compression component of a beam, or at the ends of a tension member, when side fillets are used alone, they shall have a length along each joint line at least equal to the width of the connected component. If the connected component is tapered, the length of weld shall be the greater of:
   i. the width of the widest part; and
   ii. the length of the taper
b) At the cap plate or base-plate of a compression member, welds shall have a length along each joint of at least the maximum width of the member at the contact face.
c) Where a beam is connected to the face of a compression member, the welds connecting the compression member components shall extend between the levels of the top and bottom of the beam and in addition:
d) For an unrestrained (simple) connection, a distance (d) below the lower face of the beam; and For a restrained (rigid or semi-rigid) connection, a distance (d) above and
   i. below the upper and lower faces of the beam. where d is the maximum cross-sectional dimension of the compression member.

10.12.2 Transverse spacing of fillet welds

If two parallel fillet welds connect two components in the direction of the design action to form a built up member, the transverse distance between the welds shall not exceed 32 t_p, except that in the case of intermittent fillet welds at the ends of a tension member, the transverse distance shall not exceed either 16 t_p or 200 mm, where t_p is the thickness of the thinner of the two components connected

10.13 Combination of Stresses

10.13.1 Fillet welds

10.13.1.1 When subjected to a combination of normal and shear stress, the equivalent stress f_e shall satisfy the following: $f_e = \sqrt{f_a^2 + 3q^2} \leq \frac{f_u}{\sqrt{3}\gamma_{mw}}$

where, f_a = normal stresses, compression or tension, due to axial force or bending moment
q = shear stress due to shear force or tension

10.13.1.2 Check for the combination of stresses need not be done for:

a) side fillet welds joining cover plates and flange plates, and
b) fillet welds where sum of normal and shear stresses does not exceed f_{wd} (see 10.7.4.7)

10.13.2 Groove/Butt welds

10.13.2.1 Check for the combination of stresses in butt welds need not be carried out provided that:

a) butt welds are axially loaded, and
b) in single and double bevel welds the sum of normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50 % of the design shear stress.

10.13.3 Combined bearing, bending and shear

Where bearing stress, f_{br}, is combined with bending (tensile or compressive), f_b and shear stresses, q under the most unfavorable conditions of loading in butt welds, the equivalent stress, f_e, as obtained from the following formula, shall not exceed the values allowed for the parent metal:

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2}$$

where, f_e = equivalent stress; f_b = calculated stress due to bending, in N/mm^2;
f_{br} = calculated stress due to bearing, in N/mm^2; q = shear stress, in N/mm^2;

10.14 Packing in Construction

10.14.1 Where a packing is welded between two members and is less than 6 mm thick, or is too thin to allow provision of adequate welds or to prevent bucking, the packing shall be trimmed flush with the edges of the element subject to the design action and the size of the welds along the edges shall be increased over the required size by an amount equal to the thickness of the packing.

10.14.2 Otherwise, the packing shall extend beyond the edges and shall be fillet welded to the pieces between which it is fitted.

10.15 Design of Connection

10.15.1 Each element in a connection shall be designed so that the structure is capable of resisting the design actions. Connections and adjacent regions of the members shall be designed by distributing the design action effects such that the following requirements are satisfied:

a) Design action effects distributed to various elements shall be in equilibrium with the design action effects on the connection,
b) Required deformations in the elements of the connections are within their deformations capacities,
c) All elements in the connections and the adjacent areas of members shall be capable of resisting the design action effects acting on them, and
d) Connection elements shall remain stable under the design action effects and deformation

10.15.2 Connections can be classified as rigid, semi-rigid and flexible for the purpose of analysis and design as per the recommendation in Annex H. Connections with sufficient rotational stiffness may be considered as rigid. Examples of rigid connections include flush end-plate connection and extended end-plate connections. Connections with negligible rotational stiffness may be considered as flexible (pinned). Examples of flexible connections include single and double web angle connections and header plate connections. Where a connection cannot be classified as either rigid or flexible, it shall be assumed to be semi-rigid. Examples of semi rigid connections include top and seat angle connection and top and seat angle with single/double web angles.

10.15.3 Design shall be on the basis of any rational method supported by experimental evidence. Residual stresses due to installation of bolts or welding normally need not be considered in statically loaded structures, Connections in cyclically loaded structures shall be designed considering fatigue as given Section 12. For earthquake load combinations, the connections shall be designed to withstand the calculated design action effects and exhibit required ductility as specified in Section 12.

10.15.4 Beam and column splice shall be designed in accordance with the recommendation given in annex H.

10.16 Minimum Design Action on Connection

Connections carrying design action effects, except for lacing connections, connections of sag rods, purlins and girts, shall be designed to transmit the greater of:

a) the design action in the member; and
b) the minimum design action effects expressed either as the value or the factor times the member design capacity for the minimum size of member required by the strength limit state, specified as follows:
    i) *Connections in rigid construction* - a bending moment of at least 0.5 times the member design moment capacity
    ii) *Connections to beam in simple construction* - a shear force of at least 0.15 times the member design shear capacity of $40kN$, whichever is lesser
    iii) *Connections at the ends of tensile or compression member*- a force of at least 0.3 times the member design capacity
    iv) *Splices in members subjected to axial tension*- a force of at least 0.3 times the member design capacity in tension
    v) *Splices in members subjected to axial compression*- for ends prepared for full contact in accordance with 17.7.1, it shall be permissible to carry compressive actions by bearing on contact surfaces. When members are prepared for full contact to bear at splices, there shall be sufficient fasteners to hold all parts securely in place. The fasteners shall be sufficient to transmit a force of at least 0.15 times the member design capacity in axial compression.

    When members are not prepared for full contact, the splice material and its fasteners shall be arranged to hold all parts in line and shall be designed to transmit a force of at least 0.3 times the member design compression.

    In addition, capacity splices located in axial between points of effective lateral support shall be designed for the design axial force, P_d plus a design bending moment, not less than the design bending moment: $M_d = \frac{P_d l_s}{1000}$

    where, l_s is the distance between points of effective lateral support.

    vi) *Splices in flexural members* — a bending moment of 0.3 times the member design capacity in bending. This provision shall not apply to splices designed to transmit shear force only.
    A splice subjected to a shear force only shall be designed to transmit the design shear force together with any bending moment resulting from the eccentricity of the force with respect to the centroid of the group.
    vii) *Splices in members subject to combined actions* — a splice in a member subject to a combination of design axial tension or design axial compression and design bending moment requirements shall satisfy in (4), (5) and (6) above, simultaneously.
    a. For earthquake load combinations, the design action effects specified in this section may need to be increased to meet the required behavior of the steel frame and shall comply with Section 11.

10.17 Intersection

Members or components meeting at a joint shall be arranged to transfer the design actions between the parts, wherever practicable, with their centroidal axes meeting at a point. Where there is eccentricity at joints, the members and components shall be designed for the design bending moments which result due to eccentricity.

The disposition of fillet welds to balance the design actions about the centroidal axis or axes for end connections of single angle, double angle and similar type members is not required for statically loaded members but is required for members, connection components subject to fatigue loading.

Eccentricity between the centroidal axes of angle members and the gauge lines for their bolted end connections may be neglected in statically loaded members, but shall be considered in members and connection components subject to fatigue loading.

10.18 Choice of Fasteners

Where slip in the serviceability limit state is to be avoided in a connection, high-strength bolts in a friction-type joint, fitted bolts or welds shall be used.

When a joint is subjected to impact or vibration, either high strength bolts in a friction type joint or ordinary bolts with locking devices or welds shall be used.

10.19 Connection Components

Connection components (cleats, gusset plates, brackets and the like) other than connectors, shall have their capacities assessed using the provisions of previous sections.

10.20 Analysis of a Bolt/Weld Group

10.20.1 Bolt/Weld Group Subject to In-plane Loading

10.20.1.1 General method of analysis

The design force in a bolt/weld or design force per unit length in a bolt/weld group subject to in-plane loading shall be determined in accordance with the following:

a) The connection plates shall be considered to be rigid and to rotate relative to each other about a point known as the instantaneous centre of rotation of the group.
b) In the case of a group subject to a pure couple only, the instantaneous centre of rotation coincides with the group centroid. In the case of in-plane shear force applied at the group centroid, the instantaneous centre of the rotation is at infinity and the design force is uniformly distributed throughout the group. In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the group centroid shall be superposed, or a recognized method of analysis shall be used.
c) The design force in a bolt or design force per unit length at any point in the group shall be assumed to act at right angles to the radius from that point to the instantaneous centre, and shall be taken as proportional to that radius.

10.20.2 Bolt/Weld Group Subject to Out-of-Plane Loading

10.20.2.1 General method of analysis

The design force of a bolt in bolt group or design force per unit length in the fillet weld group subject to out of-plane loading shall be determined in accordance with the following:

a) Design force in the bolts or per unit length in the fillet weld group resulting from any shear force or axial force shall be considered to be equally shared by all bolts in the group or uniformly distributed over the length of the fillet weld group.
b) Design force resulting from a design bending moment shall be considered to vary linearly with the distance from the relevant centroidal axes:
    i) In bearing type of bolt group plates in the compression side of the neutral axis and only bolts in the tension side of the neutral axis may be considered calculating the neutral axis and second moment of area.
    ii) In the friction grip bolt group only the bolts shall calculation of neutral axis and second moment of area.
    iii) The fillet weld group shall be considered in isolation from the connected element; for the calculation of centroid and second moment of the weld length.

10.20.2.2 Alternative analysis

The design force per unit length in a fillet weld/bolt group may alternatively be determined by considering the fillet weld group as an extension of the connected member and distributing the design forces among the welds of the fillet weld group so as to satisfy equilibrium between the fillet weld group and the elements of the connected member

10.20.3 Bolt/Weld Group Subject to In-plane and Out-of-Plane Loading

10.20.3.1 General method of analysis

The design force in a bolt or per unit length of the weld shall be determined by the superposition of analysis for in-plane and out-of-plane cases discussed in 10.20.1 and 10.20.2.

10.20.3.2 Alternative analysis

The design force in a bolt or per unit length in the fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and proportioning the design force per bolt or unit length in the weld group to satisfy equilibrium between the bolt/weld group and the elements of the connected member.

Force calculated in the most stressed bolt or highest force per unit length of the weld shall satisfy the strength requirements of 10.4, 10.5 and 10.7 as appropriate.

10.21 Bolts in Combination with Welds

10.21.1 Bolts shall not be considered as sharing the load in combination with welds, except in the design of shear connections on a common faying surface where strain compatibility between the bolts and welds is considered.

10.21.2 In joints with combined bolts and longitudinal welds, the strength of the connection need not be taken as less than either the strength of the bolts alone or the strength of the welds alone.

10.22 Lug Angles

10.22.1 Lug angles connecting outstanding leg of a channel-shaped member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

10.22.2 In the case of angle members, the lug angles and their connections to the gusset or other supporting member shall be capable of developing a strength not less than 20 percent in excess of the force in the outstanding leg of the member, and the attachment of the lug angle to the main angle shall be capable of developing a strength not less than 40 percent in excess of the force in the outstanding leg of the angle.

10.22.3 In the case of channel members and the like, the lug angles and their connection to the gusset or other supporting member shall be capable of developing a strength of not less than 10 percent in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member shall be capable of developing 20 percent in excess of that force.

10.22.4 In no case shall fewer than two bolts or equivalent welds be used for attaching the lug angle to the gusset or other supporting member.

10.22.5 The effective connection of the lug angle shall, as far as possible terminate at the end of the member connected, and the fastening of the lug angle to the main member shall preferably start in advance of the direct connection of the member to the gusset or other supporting member.

10.22.6 Where lug angles are used to connect an angle member, the whole area of the member shall be taken as effective notwithstanding the requirements of Section 6 of this standard.

11 Design and Detailing for Earthquake Loads

11.1 Section Classification

Structural sections of lateral load resisting system shall comply with the width-to-thickness requirements specified in Table 24. Special requirements of special moment resisting frame (SMRF), special concentrically braced frame (SCBF), and eccentrically braced frame (EBF), designed as lateral load resisting system in buildings, are provided hereunder.

11.2 Design requirement

The general requirements for ductile design and detailing of steel buildings shall ensure that required capacity is provided to cater to the imposed demand, in terms of the following broad aspects: Stability, Stiffness, Strength and Deformability and ductility. Here, the words capacity and demand refer to all the aspects specified above. Thus, steel buildings designed and detailed as per this standard are expected to resist design earthquake hazard defined in NBC 105 without collapse.

11.3 Stability

Under the action of the design loads, a building and its components shall be stable, and overall force and moment equilibrium shall be satisfied. Thus, the building shall not slide or overturn under the action of the design loads.

11.3.1 Stability Bracing Requirement for Members

When required for structural systems, stability bracing shall be provided as specified to restrain flexural or lateral-torsional buckling of steel components or members subject to axial compression, bending moment or shear force. The strength of bracing connections shall be at least 1.5 times the corresponding strength of the bracing.

Table 24: Limiting Width-to-Thickness Ratios for Compression Elements of Earthquake Resistant Structures

SN	Component	Section Type	Limiting Plate Slenderness	
			Outstanding Flange Width-to-Thickness Ratio	**Web Depth-to-Thickness Ratio**
i	Beam	Doubly symmetric rolled I-sections	$\frac{9.0\epsilon}{\sqrt{R_y}}$	$\frac{44.5\epsilon}{\sqrt{R_y}}$
		Doubly symmetric built-up I-sections		
ii	Column	Doubly symmetric rolled I-sections	$\frac{9.0\epsilon}{\sqrt{R_y}}$	$\frac{72.7\epsilon}{\sqrt{R_y}}(1-1.04C_a)$ for $C_a \leq 0.118$ and $\frac{24.9\epsilon}{\sqrt{R_y}}(2.68-C_a) \geq \frac{44.4\,\epsilon}{\sqrt{R_y}}$ for $C_a > 0.118$
		Doubly symmetric built-up I-sections		
iii	Brace	Rolled or built-up I-Section	$\frac{11.3\epsilon}{\sqrt{R_y}}$	$\frac{44.4\epsilon}{\sqrt{R_y}}$
		Closed box sections	$\frac{21.4\epsilon}{\sqrt{R_y}}$ (Flange width is the flange width minus thickness of the webs)	$\frac{21.4\epsilon}{\sqrt{R_y}}$

iv	Links	Doubly-symmetric rollled or built-up I-section	$\frac{11.3\epsilon}{\sqrt{R_y}}$	$\frac{44.4\epsilon}{\sqrt{R_y}}$
		Closed box sections	$\frac{21.4\epsilon}{\sqrt{R_y}}$ (Flange width is the flange width minus thickness of the webs)	$\frac{49.4\epsilon}{\sqrt{R_y}}$
where $\epsilon = \sqrt{\frac{230}{f_y}}$; and $C_a = \frac{P_u}{P_y/\gamma_{m0}}$				

11.4 Beam-Column Joint

At a beam–column joint, the following design aspects shall be addressed:

a) Column to beam strength ratio,
b) Joint panel zone design, and
c) Beam–column connection design.

11.4.1 Basis of Design

Flexural plastic hinges are expected to be formed at the end regions of the beams away from the column face. Under this condition, the column and the beam column joint, including the beam-column connection, is expected to remain elastic and shall be designed as capacity protected elements.

11.5 Beam-Column Connection

Fully-restrained, reinforced beam-column connections shall be used in moment frames, capable of transferring at least a bending moment of 1.1 $R_y f_y Z_{pb}$ and shear demand determined based on capacity design principle considering, (a) beams bending in double curvature, (b) plastic hinges of strength 1.1 $R_y f_y Z_{pb}$ assumed to act at a distance ⁄ from the end of the connection, and (c) gravity load required to be carried.

11.5.1 Welded Beam-Column Connection

In general, beam flanges shall be connected to column flanges using complete joint penetration groove welds, while beam web shall be connected to the column flange using either a complete joint penetration groove weld extending between weld access holes, or using a bolted single plate shear connection.

A weld access hole detail shall be adopted to ensure that the location of maximum plastic strain does not occur at the interface between the beam web and beam flange but is entirely in the beam web. Use of no weld access hole detail is not permitted. Removal of backing bar after welding and finishing of surface through grinding shall be ensured.

In general, a cover plated welded flange connection with welded rib plates at both beam flange levels is preferable.

11.6 Column Base

Column bases may have any form of embedded connection or anchor bolted base plate connection. The degree of fixity offered by a connection should be established and used in structural analysis.

11.6.1 Strength

The required design strength of the steel elements at the column base, including base plate, anchor bolts, stiffening plates, and shear lug elements shall be to resist design strength of columns over it.

11.6.2 Fixed Column Base

Fixed column base connections and supporting foundation shall be designed to resist moment demand of $1.1R_yM_{pc}$ and shear demand equal to $2.2R_yM_{pc}/H_c$, where M_{pc} and H_c are the plastic moment capacity and the clear height of the column between the connections, respectively.

11.6.3 Pinned Column Base

Instead of detailed calculations establishing rotational stiffness (based on the degree of fixity) and bending moment strength characteristics, it is permitted to analyze and design anchor bolted base plate connections at column bases in buildings as pinned connection. In such cases, the connection and supporting foundation shall be designed for minimum moment of $0.5R_yM_{yc}$,where M_{yc} is the yield moment capacity of the column section, in addition to shear force demand equal to $1.1R_yM_{pc}/H_c$.

11.7 Special Moment Resisting Frames (SMRFs)

SMRFs of structural steel shall be designed to satisfy the requirements of this section.

11.7.1 Basis of Design

SMRFs designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the beams, limited yielding of panel zones, and little or no yielding of columns except at base. SMRFs may be used except in buildings taller than 15 m. Yielding of beam to column connections in SMRFs shall not be permitted by this standard.

11.7.2 Load Combination

Columns shall be checked for the most unfavorable combination of axial force, shear force and bending moments. The column shall be checked for the following load combinations:

$$P_d = P_G + 1.1\gamma_{ov}\vartheta P_E$$

$$V_d = V_G + 1.1\gamma_{ov}\vartheta V_E$$

$$M_D = M_G + 1.1\gamma_{ov}\vartheta M_E$$

where, P_d = Design axial force
P_G = Induced axial force due to gravity loads $(DL + \lambda LL)$,
(where λ is defined in accordance with NBC 105)
V_d = Design shear force
V_E = Induced axial force due to gravity loads
P_E = Induced axial force due to earthquake loads
M_d = Design bending moment
M_G = Induced bending moment due to gravity loads $(DL + \lambda LL)$,
M_E = Induced bending moment due to earthquake loads
γ_{ov} = Material Overstrength factor = 1.25
ϑ = Design Overstrength factor= 2.5

11.7.3 Analysis

It is preferable to plan buildings to have independent planar lateral load resisting moment frames in each principal plan directions. In such cases, there are no special analysis requirements. But when two moment frames oriented in orthogonal directions intersect, the possibility of beam yielding in both orthogonal directions simultaneously shall be considered in the design of the common column.

11.7.4 System requirements

The requirements given hereunder shall be satisfied by the building system.

11.7.5 Beam column connections

Beam to column connections shall be capable of accommodating storey drift angle of 0.04 radians, without loss of strength exceeding 15 percent of the beam plastic moment capacity.

11.7.6 Column to beam strength ratio

At a beam–column joint, the following strength ratio shall be satisfied:

$$\frac{\sum M_{pc}}{\sum M_{bo}} = \frac{\sum Z_{pc} f_{yc} \left(1 - \frac{P_u}{P_d}\right)}{\sum 1.1 R_y Z_{pb} f_{yb}} \geq 1.2$$

where, Z_{pc} and Z_{pb} are the plastic section modulus and f_{yc} and f_{yb} are the characteristic yield strength of column and beam cross-sections respectively, P_u is the maximum factored axial compressive load and P_d is the design strength under axial compression, and R_y is the material uncertainty factor corresponding to the grade of steel.

11.7.7 Beams

Beams in SMRFs are permitted to carry gravity loads through composite action with reinforced concrete slab. For lateral load action, composite action shall not be considered. Further, abrupt changes in beam flanges, through actions like drilling of holes or trimming of flange width, and use of shear studs are prohibited in the beam end regions of length at least twice the depth of the beam where flexural plastic hinges are expected to be formed. Beams shall have sufficient resistance against lateral and torsional buckling.

11.7.7.1 Sections

Only doubly-symmetric parallel flange standard rolled sections or built-up sections, with flange width to thickness ratio and web depth to thickness ratio values less than the limits specified in Table 24, shall be used as beams. The flange to web weld in built-up beams shall be continuous.

11.7.7.2 Slenderness

The ratio of the maximum unbraced length of the compression flange of a beam L_{br} to the radius of gyration r_y about the weaker axis of the beam cross-section shall not exceed 25 for a distance of $2d_b$ from the end of the beam to column connection. L_{bf}/r_y for the remaining portion of the beam shall not exceed 0.10 $E/(R_y f_{yb})$.

11.7.7.3 Bracing

The stiffness of bracing shall be as given below:

a) Beams shall be restrained against rotation about their longitudinal axis at supports and at intermediate locations along the length of the beam through the use of internal panel bracing without any external rigid support.
b) The lateral bracing shall be attached at or near the compression flange of the beam.
c) The lateral bracing shall be attached at or near both flanges, near the point of inflection in beams bending in double curvature.

11.7.7.4 Stiffness of Bracing

The stiffness of bracing shall be as given below:

a) The shear stiffness of the panel bracing system closest to the inflection point in a beam bending in double curvature shall be: $K_{br} \geq \frac{10\, R_y M_{pb}}{L_{br} d_f}$ where L_{br} is the unbraced length; and d_f is the distance between centroids of the flanges of the beam.
b) The shear stiffness of the panel bracing system other than near the inflection point in a beam bending in double curvature shall be: $K_{br} \geq \frac{5\, R_y M_{pb}}{L_{br} d_f}$

11.7.7.5 Strength of Bracing

The shear strength of the panel bracing system shall be: $V_{br} \geq 0.025 \frac{R_y M_{pb}}{d_f}$

11.7.7.6 Special Bracing at Plastic Hinge Locations

Special bracing shall be located adjacent to the expected plastic hinge locations. Both flanges of beams shall be laterally braced. The axial strength of such lateral bracing shall be $P_{br} \geq 0.06 \frac{R_y M_{pb}}{d_f}$ and the required bracing stiffness shall be as in 11.7.7.4(b).

11.7.7.7 Strength

The design strength of beam shall satisfy the load combinations in NBC 105, and the overstrength load combinations specified in 11.8.2 and 11.9.2 in SCBFs and EBFs respectively, or when a beam is part of diaphragm collector or chord.

11.7.7.8 Shear Strength

The design shear strength of the beam at the location of the plastic hinge shall be determined as per Section 8, and it shall be at least equal to the shear demand specified in 11.8.5.5.

11.7.7.9 Beam Splice

Beam splices shall be located at least $3d_b$ away from the face of the column or d_b from the line of action of any concentrated force acting on the beam. The design strength of beam splices shall at least be 1.80 times the required strength, except at beam-column connections. Further, design strength of each flange splice plate shall at least be 1.2 $R_y f_y A_f$, where A_f is the area of the flange being spliced.

11.7.8 Columns

11.7.8.1 Sections

Only doubly-symmetric parallel flange standard rolled built-up sections, with flange width to thickness ratio, and web depth to thickness ratio less than the corresponding limits specified in Table 24, shall be used as columns. The flange to web weld in built-up beams shall be continuous.

11.7.8.2 Slenderness

The slenderness ratio of unbraced length of columns shall be less than 75.

11.7.8.3 Bracing

Columns shall be laterally braced at supports and at intermediate locations along the length of the column through the use of internal panel bracing without any external rigid support.

11.7.8.4 Stiffness of Bracing

The shear stiffness of the panel bracing system, in the direction perpendicular to the longitudinal axis of the columns shall be:

$$K_{br} \geq 3 \frac{P_u}{L_{br}}$$

where, P_u = Maximum factored axial load; and L_{br} = Unbraced length of the column

11.7.8.5 Strength of Bracing

The shear strength of the panel bracing system shall at least be: $V_{br} \geq 0.005 \, P_u$.

11.7.8.6 Strength of Bracing Connection

The connection of the bracing system to the column shall have design strength at least equal to $P_{br} \geq 0.01 \, P_u$ subject to minimum design action on connection given in section 10.

11.7.8.7 Strength

Columns shall have design strength more than the maximum demand arising from the following:

a) Structural analysis based on load combinations specified in 3.3, and
b) Maximum loads transferred to the column considering 1.2 times the strength of the connected members (beams, braces, etc) determined considering material strength uncertainty factor.

Columns in both moment frames and braced frames that are common to intersecting frames aligned along two orthogonal directions, shall consider in design the potential for simultaneous inelasticity from all such frames for determination of the required axial strength, including the overstrength earthquake load or the capacity-limited earthquake load, as applicable.

The direction of application of the load in each such frame shall be selected to produce the most severe load effect on the column.

Columns in buildings designed to resist lateral loads shall not carry tensile forces.

11.7.9 Splice

Column splices shall be located in the middle third of the height of the columns, at least 1.0 m away from the beam-to-column moment connection. The design strength demand of column splices shall at least be that determined using 11.8.2. Further, the design strength of both flange and web splice plates shall at least be 1.2 R_y times of their respective strengths.

11.7.10 Joint panel zone

Shear yielding of joint panel zone (JPZ) shall be limited. Use of continuity and doubler plates is permitted Figure 25.

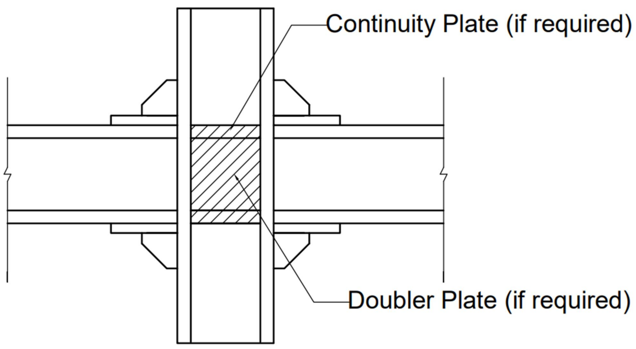


Figure 25: Typical interior reinforced Beam-Column joint

11.7.10.1 Panel Zone Demand

The shear force demand at the face of the column flanges shall be taken as $V_{pzd} = \sum \frac{1.1 R_y f_y Z_{pb}}{0.95\, d_b}$

11.7.10.2 Panel Zone Capacity

The design shear strength capacity shall be: $V_{pzc} = \frac{f_y}{\sqrt{3}\gamma_{m0}}(0.95\, d_c) t_{pz}$

where, t_{pz} = thickness of the panel zone, including thickness of doubler plate if provided; and
γ_{m0} =1.1;

11.7.10.3 Panel Zone Thickness

The individual thickness of the column web and doubler plates (when provided), shall be more than:

$$(d_{pz} + w_{pz})/\ 90$$

where, $d_{pz} = d_p - 2t_{bf}$ of the deeper beam, in mm; and $w_{pz} = d_c - 2t_{cf}$, in mm.

11.7.10.4 Doubler Plate

Doubler plates with plug welding shall be provided when the thickness of the column web within the panel does not satisfy the strength requirements. It is permitted to use doubler plates with or without continuity plates. When continuity plates are not provided, doubler plates shall extend at least 150 mm beyond the deeper beam flange levels on either sides of the panel zone.

11.7.10.5 Continuity Plate

Continuity plates shall be provided when: $\frac{6.25\ f_y t_{cf}^2}{1.1} \leq 1.2\ R_y f_y b_{bf} t_{bf}$

When provided, thickness of continuity plate shall not be less than thickness of the thinner beam flange on either side of the column, and width not less than the distance of the tip of the wide beam flange from the face of the column web. If different grades of steel are used, appropriate values of f_y shall be used on either side of the inequality.

11.7.11 Protected zones

The region at each end of a beam equal to twice the depth of the beam subjected to inelastic straining shall be designated as a protected zone. Steel headed stud anchors and other fabrication and erection attachments shall not be placed on beam flanges within the protected zone.

11.7.12 Demand critical welds

The following welds shall be designed as demand critical welds:

a) Groove welds at column splices;
b) Welds at column-to-base plate connections, except when:
    i) column plastic hinge at, or near the base plate is precluded by conditions of restraint, or
    ii) there is no net tension under load combinations including over-strength earthquake load;
c) Welds in beam-column connections.

11.8 Special Concentrically Braced Frames (SCBFs)

SCBFs of structural steel shall be designed to satisfy the requirements given hereunder. Collector beams that connect SCBF braces shall be considered to be part of SCBF.

11.8.1 Basis of Design

In SCBFs, members shall be concentrically connected. But eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design, and the eccentricities do not change the expected source of inelastic deformation capacity of the building. SCBF designed in accordance with these provisions are expected to provide inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.

11.8.2 Load Combinations

11.8.2.1 Beams and columns with axial forces should meet the following minimum resistance requirements:

$$P_D = P_G + 1.1\gamma_{ov}\vartheta P_E$$

$$M_D = M_G + 1.1\gamma_{ov}\vartheta M_E$$

where, P_D = Design axial force;
P_G = Induced axia l force due to gravity loads $(DL + \lambda LL)$;
P_E = Induced axial force due to earthquake loads;
M_D = Design bending moment;

M_G = Induced bending moment due to gravity loads $(DL + \lambda LL)$;
M_E = Induced bending moment due to earthquake loads;
γ_{ov} = Material Overstrength factor = 1.25;
ϑ = Design Overstrength factor = 2.0

11.8.3 Analysis

The following shall be satisfied in the analysis of SCBFs:

11.8.3.1 The required strength of braces shall be determined based on the analysis required by NBC 105. Further, the required strength of braces shall not exceed the design strength in case of pure axial compression.

11.8.3.2 The required strength of capacity-protected elements (columns, beams, struts, collectors and connections) shall be taken as the larger force determined from the following analysis:

a) An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension;
b) An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength; and
c) For multi-tiered braced frames, analyses representing progressive yielding and buckling of the braces from weakest tier to strongest. Analyses shall consider both directions of frame loading.

The expected tensile strength (T_e) of braces in tension shall be taken as: $T_e = R_y f_y A_g$

The expected compressive strength (P_e) of the braces shall be taken as: $P_e = R_y \gamma_{m0} P_d$

where, P_d is the design compressive strength as determined 7.1.2. The expected post-buckling strength of the braces in compression shall be taken as 0.2 times the expected compressive strength (P_e) and the bracing connections shall be assumed to remain elastic.

11.8.4 System Requirement

The following system requirements shall be satisfied.

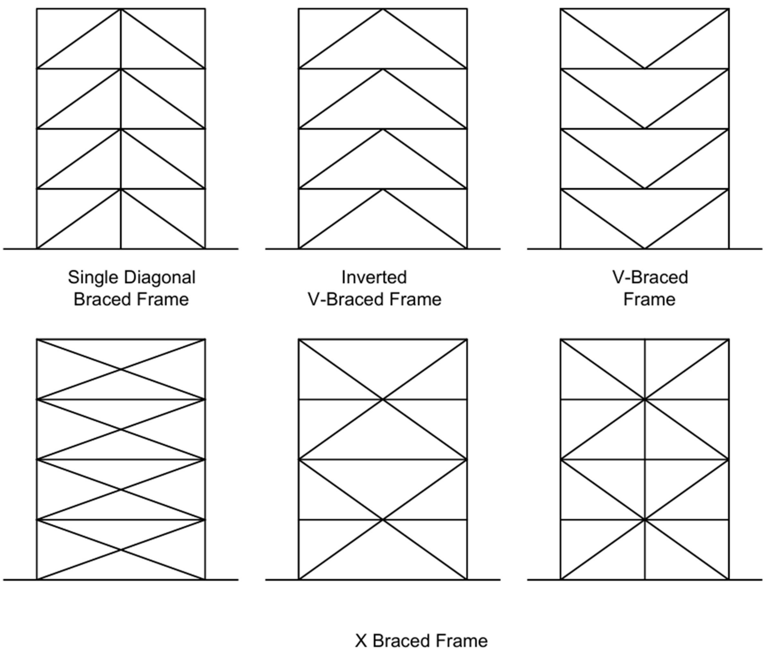


Figure 26: Concentrically Braced Frame

11.8.4.1 Diagonal and X-braced frames

Diagonal and X-braced frames are permitted to be used in SCBF.

11.8.4.2 V- and inverted V-braced frame

V- and inverted V-braced frames are permitted to be used in SCBF. In such systems, the beams that are intersected by braces away from beam-to-column connections shall satisfy the following requirements:

a) Beams shall be continuous between columns and adequately braced to prevent lateral torsional buckling; and
b) As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) braced frames, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

11.8.4.3 Continuity of load path

For the purpose of this standard, a line of braces 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 braces interconnected adequately through rigid diaphragm. A diaphragm shall be considered to be rigid if the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is less than 1.2 times the average displacement of the entire diaphragm.

11.8.4.4 Lateral force distribution

Braces shall be provided in alternate directions along each line of braces. Along any line of bracing, braces shall be provided such that for lateral loading in either direction, tension braces resist between 30 percent to 70 percent of the total horizontal load.

11.8.4.5 Multi-tiered braced frames

Multi-tiered braced frames (MT-BFs) consist of multiple vertically oriented bracing panels that lack intersecting perpendicular framing or diaphragms at the levels between the bracing panels. A special concentrically braced frame is permitted to be configured as a multi-tiered braced frame (MT-SCBF) when the following requirements are satisfied:

a) Braces shall be used in opposing pairs at every tier level.
b) Struts shall satisfy the following requirements:
    1) Horizontal struts shall be provided at every tier level;
    2) Struts that are intersected by braces away from strut-to-column connections shall also meet the requirements stated in 11.7.3. When brace buckling occurs out-of-plane, torsional moments arising from brace buckling shall be considered when verifying lateral bracing or minimum out-of-plane strength and stiffness requirements. The torsional moments shall correspond to $1.1R_yM_p$ of the brace about the critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connection, where M_p is the plastic bending moment.
c) Columns shall satisfy the following requirements:
    1) Columns shall be torsionally braced at every strut-to-column connection location.
    2) Columns shall have sufficient strength to resist forces arising from brace buckling. These forces shall correspond to $1.1R_yM_p$ of the brace about the critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connections.
    3) For all load combinations, columns subjected to axial compression shall be designed to resist bending moments due to second-order and geometric imperfection effects. As a minimum, imperfection effects are permitted to be represented by an out-of-plane horizontal notional load applied at every tier level and equal to 0.006 times the vertical load contributed by the compression brace intersecting the column at the tier level.

4) Lateral drift in each tier in a multi-tiered concentrically braced frame shall not exceed 0.4 percent of the tier height.

11.8.4.6 K-braced frames shall not be used in SCBF.

11.8.5 Member Requirements

The requirements specified hereunder shall be satisfied by the component or member.

11.8.5.1 Sections

Columns, beams, braces and struts in multi-tiered concentrically braced in shall comply with the width-to-thickness requirements specified in Table 24.

11.8.5.2 Braces

Structural braces may be used to impart lateral stiffness and strength to building frames. Such braces shall be provided in selected bays over the full height of the building frame. Braced part of CBF shall also conform to the requirements given hereunder.

11.8.5.2.1 Sections

Standard rolled or built-up sections or closed box sections, with flange width to thickness ratio and web depth to thickness ratio less than the limits specified in Table 24, shall be used as braces. The weld between the elements of built-up section shall be continuous.

11.8.5.2.2 Slenderness

The effective slenderness ratio of braces shall be less than 160. In case of built-up braces, at least two connectors shall be provided at uniform spacing such that the slenderness ratio of individual plate elements between the connectors shall be less than 0.4 times the governing effective slenderness ratio of the built-up brace.

11.8.5.2.3 Effective Area

Brace effective net area shall not be taken less than the gross cross-sectional area of the brace. Where reinforcement on braces is used, the following requirements shall apply:

a) The characteristic yield strength of the reinforcement, when provided in the form of steel plates, shall be at least equal to the characteristic yield strength of the brace, and

b) The connections of the reinforcement to the brace shall have sufficient strength to develop the expected reinforcement strength.

11.8.5.2.4 Bracing Connection

The required strength in tension, compression, and flexure of brace connections (including beam-to column connections if part of the braced-frame system) shall be determined as required in the following. These required strengths are permitted to be considered independently without interaction.

11.8.5.2.5 Tensile Strength

The tensile strength of brace connections shall at least be the lesser of the following:

a) The expected yield strength in tension of the brace, determined as maximum of $1.1R_y f_y A_g$ and $R_u f_u A_n$; and
b) The maximum load effect, indicated by analysis as in 5.6, that can be transferred to the brace by the system.

11.8.5.2.6 Compressive Strength

The compressive strength of brace connections shall at least be equal to the brace strength in compression, generally as governed by buckling.

11.8.5.2.7 Accommodation of Brace Buckling

Brace connections shall be designed to withstand the flexural forces and rotations imposed by brace buckling. Connections satisfying the following provisions are deemed to satisfy this requirement:

a) Required Flexural Strength: Brace connections designed to withstand the flexural forces imposed by brace buckling shall have a required flexural strength equal to 1.1 times the expected brace flexural strength. The expected brace flexural strength shall be determined as $R_y f_y Z_p$ of the brace about the critical buckling axis.

b) Rotation Capacity: Brace connections shall be designed to withstand the rotations imposed by brace buckling. Inelastic rotation of the connection is permitted.

11.8.5.2.8 Gusset Plates

To accommodate brace buckling, gusset plates shall be detailed to undergo out-of-plane bending and welds that attach a gusset plate directly to a beam flange or column flange shall be designed to have shear strength per unit length equal to $\frac{R_y f_y t_p}{\sqrt{3}}$, where t_p is the thickness of the gusset plate.

11.8.5.3 Protected Zones

The protected zone of SCBF shall satisfy 11.2 and shall include the following:

1. For braces, the centre one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling; and
2. Elements that connect braces to beams and columns.

11.8.5.4 Beams

In addition to satisfying requirements of beams for special moment resisting frame, beams shall be checked for axial load arising due to analysis case 11.7.3.

11.8.5.5 Beam-to-column connections

Where a brace-gusset plate assembly connects to both members at a beam-to-column connection, the connection assembly shall be designed to resist beam moment taken equal to $1.1R_y f_{yb} Z_{pb}$. Also, the sum of the expected column flexural strengths shall exceed $1.1R_y f_{yb} Z_{pb}$.

This connection moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the over-strength earthquake load.

11.8.5.6 Column Splices

Requirements specified in 11.7.9 shall apply. In addition, column splices shall be designed to develop at least 50 percent of the plastic flexural strength, M_p, of the smaller connected member, and have shear strength greater than $\sum M_p / H_c$:

where, $\sum M_p$ = Sum of the plastic flexural strengths, at the top and bottom ends of the column; and
H_c = clear height of the column between beam connections.

11.8.5.7 Demand Critical welds

Positions of demand critical welds are the same as that mentioned for special moment resisting frames.

11.9 Eccentrically Braced Frames (EBFs)

EBF of structural steel shall be designed to satisfy the requirements given hereunder.

11.9.1 Basis of Design

These provisions are applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centrelines of the beam and an adjacent brace or column, forming a link that is, subject to shear and flexure (see Figure 27).

Eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear (or flexural) yielding in the links. Links shall not be connected directly to columns.

11.9.2 Load and Load combinations

11.9.2.1 The members, if horizontal links in beams are used, and also the beam members, if vertical links are used, should be verified in compression considering the most unfavorable combination of the axial force and bending moments:

$$V_d = V_G + 1.1\gamma_{ov}\vartheta V_E$$

$$M_D = M_G + 1.1\gamma_{ov}\vartheta M_E$$

where, V_d = Design shear force;
V_G = Induced axial force due to gravity loads $(DL + \lambda LL)$;
M_d = Design bending moment;
M_G = Induced bending moment due to gravity loads $(DL + \lambda LL)$;
M_E = Induced bending moment due to earthquake loads;
γ_{ov} = Material Overstrength factor = 1.25;
ϑ = Design Overstrength factor=2.0

11.9.3 Analysis

The requirements specified hereunder shall be satisfied in the analysis of EBFs.

11.9.3.1 The shear demand on the links shall be determined based on the analysis required by NBC 105.

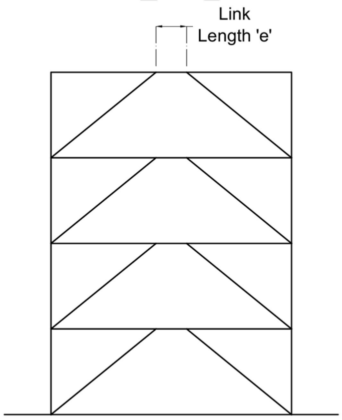


Figure 27: Eccentrically Braced Frame

11.9.3.2 The required strength of capacity-protected elements (columns, beams, diagonal braces and connections) shall be determined based on the expected over-strength capacity of the link to be taken as $1.1R_yS_h$ times the design strength of the link as per 11.9.5.2, where S_h (a factor to account for strain hardening and strain rate) is equal to 1.25 for I-shaped links and 1.4 for box shaped links.

Exception: When the link and beam have the same section and is continuous:

a) the required strength of the beam outside the link shall be determined based on 0.9 times the expected link over strength capacity; and
b) the design capacity of the beam shall be calculated based on the expected material yield stress.

11.9.4 System Requirements

The requirements specified hereunder shall be satisfied.

11.9.4.1 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. The link rotation angle shall not exceed 0.08 rad.

11.9.4.2 Bracing of link

Bracing shall be provided at both the top and bottom flanges of the link at the ends of the link for I-shaped sections. Such bracings shall have stiffness and strength as specified in 11.7.7.6.

11.9.5 Member Requirements

The requirements specified hereunder shall be satisfied.

11.9.5.1 Basic requirements

Brace members, beams outside the links and columns shall satisfy width-to-thickness limitations specified in Table 24. Apart from columns, the beams and braces in EBFs may be subjected to significant axial and bending forces; hence their design capacities shall be determined as for beam-column members.

11.9.5.2 Shear Links

Shear links may be used as structural fuse in EBFs as specified in this standard. These links are expected to be subjected to combined action of bending moment and shear force and undergo yielding under earthquake effects.

Sections

Links shall be I-shaped cross sections (standard rolled wide-flange sections or built-up sections), or built-up box sections, with flange width to thickness ratio and web depth to thickness ratio less than the limits specified in Table 2. Hollow structural steel (HSS) sections shall not be used as links. Further:

a) The web or webs of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted;
b) For links made of built-up cross sections, complete-joint-penetration groove welds shall be used to connect the web (or webs) to the flanges; and
c) Links of built-up box sections shall have a moment of inertia, I_y, about an axis in the plane of the EBF greater than $0.67 I_x$, where I x is the moment of inertia of the link about an axis perpendicular to the plane of the EBF.

Link shear strength

The design shear strength of link shall be the lower value of the following:

a) For shear yielding: $\frac{V_{pL}}{\gamma_{m0}}$

where, $V_{pL} = \frac{f_y A_{wL}}{\sqrt{3}}$ for $\frac{P_u}{P_y} \leq 0.15$

$$V_{pL} = \frac{f_y A_{wL}}{\sqrt{3}} \sqrt{(1 - \left(\frac{P_u}{P_y}\right)^2} \text{ for } \frac{P_u}{P_y} > 0.15$$

$A_{wL} = (d_L - 2t_f)t_w$ for I-shaped link sections, $= 2(d_L - 2t_f)t_w$ for box link sections;

$P_u =$ Factored axial load in the link;

$P_y = f_y A_{gl}$; $A_{gL} =$ Gross cross-sectional area of link;

$d_L =$ Overall depth of link; $t_f =$ Thickness of flange; $t_w =$ Thickness of web; and

b) For flexural yielding: $\frac{2M_{pL}}{e\,\gamma_{m0}}$

where, $M_{pL} = f_y Z_{pL}$ for $\frac{P_u}{P_y} \le 0.15$

$$M_{pL} = f_y Z_{pL}\left[1 - \frac{P_u}{P_y}\right] \text{ for } \frac{P_u}{P_y} > 0.15$$

$Z_{pL} =$ Plastic section modulus of link about the bending axis.

Length of link

The length of link e shall be less than $1.6\frac{M_{pL}}{V_{pL}}$, where M_{pL} and V_{pL} are the plastic bending moment capacity and plastic bending moment capacity and plastic shear capacity of the link. Further, the following shall be satisfied:

If $\frac{P_u}{P_y} > 0.15$, the length of the link shall be limited as follows:

$$e \le 1.6\frac{M_{pL}}{V_{pL}} \text{ for } \rho^- \le 0.5\,; \qquad e \le 1.6\frac{M_{pL}(1.15-0.3\rho^-}{V_{pL}} \text{ for } \rho^- > 0.5$$

where, $\rho^- = \frac{P_u/P_y}{V_u/V_y}$, and $V_y = \frac{f_y A_{wL}}{\sqrt{3}}$

Stiffeners for I-shaped Link Sections

Web of links shall be stiffened to prevent premature buckling. The required strength of fillet welds connecting a stiffener to the link web shall be $f_y A_{st}$, where A_{st} is the horizontal cross-sectional area of the link stiffener. The required strength of fillet welds connecting a stiffener to the link flanges is $\frac{f_y A_{st}}{4}$.

End Web 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 the larger of $0.75t_w$ or 10 mm, where b_f and t_w are the link flange width and link web thickness, respectively.

Intermediate Web Stiffeners

Links shall be provided with intermediate web stiffeners spaced at intervals not exceeding $30t_w - 0.2d$.

Stiffeners for Box Link Sections

Web of links shall be stiffened to prevent premature buckling. The required strength of fillet welds connecting a stiffener to the link web shall be $f_y A_{st}$, where A_{st} is the horizontal cross-sectional area of the link stiffener.

End Web Stiffeners

Full-depth web stiffeners shall be provided on one side of each link web at the diagonal brace connection. These stiffeners are permitted to be welded to the outside or inside face of the link webs. These stiffeners shall each have a width not less than $\frac{b}{2}$, where b is the inside width of the box section. These stiffeners shall each have a thickness not less than the larger of $0.75t_w$ or 10 mm.

Intermediate Web Stiffeners

Box links shall be provided with full-depth intermediate web stiffeners welded either to the outside or inside face of the link webs as follows:

a) When web depth-to-thickness ratio is greater than $19\epsilon/\sqrt{f_y}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20\mathrm{tw} - (\mathrm{d} - 2\mathrm{t_f})/8$; and
b) When web depth-to-thickness ratio is less than or equal to $19\epsilon/\sqrt{f_y}$, no intermediate web stiffeners are required.

11.9.5.3 Protected zones

The protected zones of EBFs are the links. Use of shear studs on the links is prohibited.

11.9.5.4 Beam-to-column connections

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection assembly shall be designed to resist beam moment taken equal to $1.1R_y f_{yb} Z_{pb}$. Also, the sum of the expected column flexural strengths shall exceed $1.1R_y f_{yb} Z_{pb}$.

This connection moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength earthquake load.

11.9.5.5 Braces

Braces in EBFs shall be designed not to yield in tension or buckle in compression corresponding to shear force in link taken equal to $1.2R_y$ times the design strength of connected link as per 11.9.5.2.

11.9.5.6 Brace Connections

When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the earthquake load effect determined using the over-strength earthquake load specified in 5.5. Connections of braces designed to resist a portion of the link end moment shall be designed as fully restrained.

11.9.5.7 Column splices

Requirements of column splices mentioned in special moment resisting frame shall apply.

In addition, column splices shall be designed to develop at least 50 percent of the plastic flexural strength, M_p, of the smaller connected member, and have shear strength greater than $\sum M_p / H_c$.

where, $\sum M_p =$ Sum of the plastic flexural strengths, at the top and bottom ends of the column; and
$H_c =$ Clear height of the column between beam connections.

11.9.5.8 Demand critical welds

The following welds shall be designed as demand critical welds:

a) Groove welds at column splices;
b) Welds at column-to-base plate connections, except when:
    i) Column plastic hinge at, or near the base plate is precluded by conditions of restraint, or
    ii) There is no net tension under load combinations including the overstrength earthquake load;
c) Welds in beam-column connections;
d) Connections of braces designed to resist a portion of the link end moment; and
e) In built-up beams, welds within the link connecting the webs to the flanges.

12 Fatigue

12.1 General

Structure and structural elements subject to loading that could lead to fatigue failure shall be designed against fatigue as given in this section. This shall however not cover the following: Corrosion fatigue, Low cycle (high stress) fatigue, Thermal fatigue, Stress corrosion cracking, Effects of high temperature ($> 150°C$), and Effects of low temperature ($<$ brittle transition temperature).

12.1.1 For the purpose of design against fatigue, different details (of members and connections) are classified under different fatigue class. The design stress range corresponding to various number of cycles, are given for each fatigue class. The requirements of this section shall be satisfied with, at each critical location of the structure subjected to cyclic loading, considering relevant number of cycles and magnitudes of stress range expected to be experienced during the life of the structure.

12.2 Design

12.2.1 Reference Design Condition

The standard S-N curves for each detail category are given for the following conditions:

a) Detail is located in a redundant load path, wherein local failure at that detail alone will not lead to overall collapse of the structure.
b) Nominal stress history at the local point in the detail is estimated/evaluated conventional by a method without taking into account the local stress concentration effects due to the detail.
c) Load cycles are not highly irregular.
d) Details are accessible for and subject to regular inspection.
e) Structure is exposed to only mildly corrosive environment as in normal atmospheric condition and suitably protected against corrosion (pit depth < 1 mm).
f) Structure is not subjected to temperature exceeding 150 °C.
g) Transverse fillet or butt weld connects plates of thickness not greater than 25 mm.

The values obtained from the standard S-N curve shall be modified by a capacity reduction factor μ_r. when plates greater than 25 mm in thickness are joined together by transverse fillet or butt welding, given by:

$$\mu_r = \left(\frac{25}{t_p}\right)^{0.25} < 1.0$$

where, t_p = actual thickness of the thicker plate being joined, in mm.

No thickness correction penetration is necessary butt weld reinforcements when full are machined flush and proved free of defect through non-destructive testing.

12.2.2 Design Spectrum

12.2.2.1 Stress evaluation

Design stress shall be determined by elastic analysis of the structure to obtain stress resultants and the local stresses may be obtained by a conventional stress analysis method. The normal and shear stresses shall be determined considering all design actions on the members, but excluding stress concentration due to the geometry of the detail. The stress concentration effect is accounted for in detail category classification (see **Table 26**). The stress concentration, however, not characteristic of the detail shall be accounted for separately in the stress calculation.

In the fatigue design of trusses made of members with open sections, in which the end connections are not pinned, the stresses due to secondary bending moments shall be taken into account, unless the slenderness ratio of the member is greater than 40.

In the determination of stress range at the end connections connection between hollow sections, the effect of stiffness disregarded, provided:

a) the calculated stress range is multiplied by appropriate factor given in **Table 24(a)** in the case of circular hollow section connections and **Table 24(b)** in the case of rectangular hollow section connections
b) the design throat thickness of fillet welds in the joints is greater than the wall thickness of the connected member

12.2.2.2 Design stress spectrum

In the case of loading events producing non-uniform stress range cycle, the stress spectrum may be obtained by a rational method, such as 'rain flow counting' or an equivalent method.

Table 25 Multiplying Factors for Calculated Stress Range (Circular Hollow Sections)

SN	Types of Connection		Chords	Verticals	Diagonals
i	Gap Connections	K type	1.5	1.0	1.3
		N type	1.5	1.8	1.4
ii	Overlap Connections	K type	1.5	1.0	1.2
		N type	1.5	1.65	1.25

Table 26: Multiplying Factors for calculated stress range (Rectangular Hollow Sections)

SN	Types of Connection		Chords	Verticals	Diagonals
i	Gap Connections	K type	1.5	1.0	1.5
		N type	1.5	2.2	1.6
ii	Overlap Connections	K type	1.5	1.0	1.3
		N type	1.5	2.0	1.4

12.2.3 Partial Safety Factors

12.2.3.1 Partial safety factor for actions and their effects (γ_{fft})

Unless and otherwise the uncertainty in the estimation of the applied actions and their effects demand a higher value, the partial safety factor for loads in the evaluation of stress range in fatigue design shall be taken as 1.0.

12.2.3.2 Partial safety factor for fatigue strength (γ_{mft})

Partial safety factor for strength is influenced by consequences of fatigue damage and level of inspection capabilities.

12.2.3.3 Based on consequences of fatigue failure, component details have been classified as given inTable 27 and the corresponding partial safety factor for fatigue strength shall be used:

a) Fail-safe structural component/detail is the one where local failure of one component due to fatigue crack does not result in the failure of the structure due to availability of alternate load path (redundant system)
b) Non-fail-safe structural component/detail is the one where local failure of one component leads rapidly to failure of the structure due to its non-redundant nature.

Table 27: Partial Safety Factors for Fatigue Strength (γ_{mft})

SN	Inspection and Access	Consequence of Failure	
		Fail-Safe	***Non-fail-Safe***
i)	Periodic inspection, maintenance and accessibility to detail is good	1.00	1.25
ii)	Periodic inspection, maintenance and accessibility to detail is poor	1.15	1.35

12.3 Detail Category

Tables 26 (a) to (d) indicate the classification of different details into various categories for the purpose of assessing fatigue strength. Details not classified in the table may be treated as the lowest detail category of a similar detail, unless superior fatigue strength is proved by testing and/or analysis.

Holes in members and connections subjected to fatigue loading shall not be made:

a) using punching in plates having thickness greater than 12 mm unless the holes are sub punched and subsequently reamed to remove the affected material around the punched hole, and
b) using gas cutting unless the holes are reamed to remove the material in the heat affected zone.

12.4 Fatigue Strength

The fatigue strength of the standard detail for the normal or shear fatigue stress range, not corrected for effects discussed in **13.2.1**, is given below **(see also Fig. 22 and Fig. 23):**

a. Normal stress range

$$\text{when } N_{SC} \leq 5 \times 10^6,\ f_t = f_{fn}\left(5 \times \frac{10^6}{N_{SC}}\right)^{\frac{1}{3}}$$

$$\text{when } 5 \times 10^6 \leq N_{SC} \leq 10^8,\ f_f = f_{fn}\left(5 \times \frac{10^6}{N_{SC}}\right)^{\frac{1}{5}}$$

b. Shear stress

$$\tau_f = \tau_{fn}\left(5 \times \frac{10^6}{N_{SC}}\right)^{\frac{1}{5}}$$

Where,

f_f, τ_f =design normal and shear fatigue stress range of the detail, respectively, for life cycle of N_{SC}, and

f_{fn}, τ_{fn} =normal and shear fatigue strength of the detail for 5 x 10”cycles, for the detail category **(see Table 26).**

Table 26: Detail Category Classification, Group 1 Non-welded Detail

12.5 Fatigue Assessment

The design fatigue strength for N_{SC} life cycles (f_{fd}, τ_{fd}) may be obtained from the standard fatigue strength for Nsc cycles by multiplying with correction factor, μ_r, for thickness, as mentioned in **13.2.1** and dividing by partial safety factor given in **Table 25.**

12.5.1 Stress Limitations

12.5.1.1 The maximum (absolute) value of the normal and shear stresses shall never exceed the elastic limit (f_y, τ_y) for the material under cyclic loading.

12.5.1.2 The maximum stress range shall not exceed $1.5\ f_y$ for normal stresses and $1.5\ f_y/\sqrt{3}$ for the shear stresses under any circumstance.

12.5.1.3 Constant stress range

The actual normal and shear stress range f and τ at a point of the structure subjected to N_{sc} cycles in life shall satisfy.

$$f \leq f_{fd} = \frac{\mu_r f_f}{\gamma_{mft}}$$

$$\tau \leq \tau_{fd} = \mu_r \tau_f / \gamma_{mft}$$

where, μ_r = correction factor **(see 13.2.1)**

γ_{mf} = partial safety factor against fatigue failure, given in **Table 25**, and

f_f, τ_f =normal and shear fatigue strength ranges for the actual life cycle, N_{SC}, obtained from 13.4

12.5.1.4 Variable stress range

Fatigue assessment at any point in a structure, wherein variable stress ranges jfi or 7$ for ni number of cycles (i =1 to r) are encountered, shall satisfy the following:

a) For normal stress (f):

$$\frac{\sum_{i=1}^{r_5} n_i f_i^3}{5\times 10^6\left(\frac{\mu_r f_{fn}}{\gamma_{mft}}\right)^3} + \frac{\sum_{j=r_5}^{r} n_j f_j^5}{5\times 10^6\left(\frac{\mu_r f_{fn}}{\gamma_{mft}}\right)^5} \le 1.0$$

b) For shear stress (τ):

$$\sum_{i=1}^{r} n_i \tau_i^5 \le 5\times 10^6\left(\frac{\mu \tau_{fn}}{\gamma_{mft}}\right)^5$$

where r_5 is the summation upper limit of all the normal stress ranges(f_i), having magnitude lesser than $\left(\frac{\mu_r f_{fn}}{\gamma_{mft}}\right)$ for that detail and the lower limit of all the normal stress ranges (f_j) having magnitude greater than $(\mu_r f_{fn}/\gamma_{mft})$ for the detail. In the above summation all normal stress ranges, f_i, and shear stress $\tau_i$having magnitude less than $0.55\ \mu_r f_{fn}$, and $0.55\mu_r \tau_{fn}$ respectively may be disregarded.

12.6 Necessity for Fatigue Assessment

a) Fatigue assessment is not normally required for building structures except as follows:
   i) Members supporting lifting or rolling loads,
   ii) Member subjected to repeated stress cycles from vibrating machinery,
   iii) Members subjected to wind induced oscillations of a large number of cycles in life, and
   iv) Members subjected to crowd induced oscillations of a large number of cycles in life.

b) No fatigue assessment is necessary if any of following conditions is satisfied.
   i) The highest normal stress range $f_{f,Max}$ satisfies$f_{f,Max} \le 27\ \mu_r/\gamma_{mft}$
   ii) The highest shear stress range $\tau_{f,Max}$ satisfies $\tau_{f,Max} \le 67\ \mu_r/\gamma_{mft}$
   iii) The total number of actual stress cyclesN_{SC}satisfies: $N_{SC} \le 5\times 10^6\left(\frac{27\mu_r}{\gamma_{mft} f_{feq}}\right)^3$,satisfies

   where, f_{feq} = equivalent constant amplitude stress range in MPa given by:

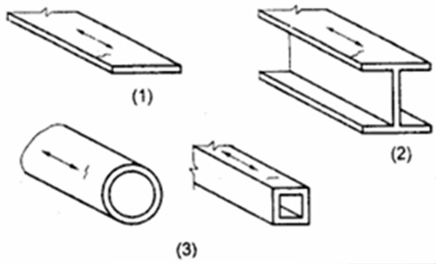
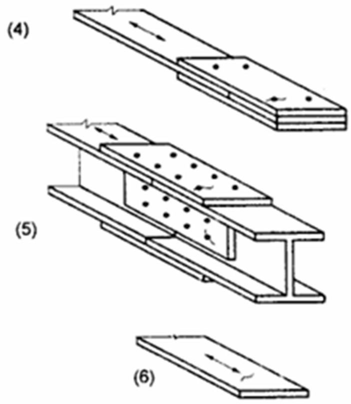
$$f_{feq} = \left[\frac{\sum_{i=1}^{r_5} n_i f_i^3 + \sum_{j=r_5}^{r} n_j f_j^5}{n}\right]^{\frac{1}{3}}$$

   where, $n = \sum_{i=1}^{r} n_i$;

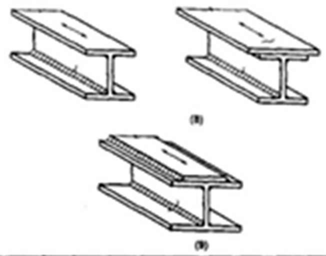
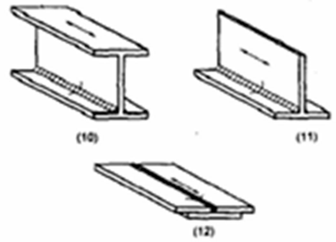
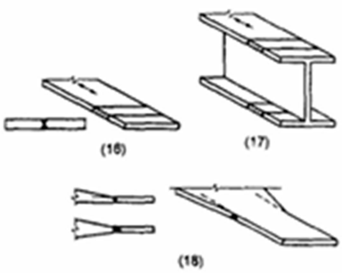
   f_i, f_j = stress ranges falling above and below the f_{fn}, the stress range corresponding to the detail at 5×10^6 number of life cycles.

   r_5 = summation upper limit of all the normal stress ranges (f_i) having magnitude lesser than ($\mu_r\ f_{fn}\ /\gamma_{mft}$) for that detail and the lower limit of all the normal stress ranges (f_j) having magnitude greater than ($\mu_r\ f_{fn}\ /\gamma_{mft}$) for the detail.

Table 26 (a) Detail Category Classification, Group 1 Non-welded Details
(*Clauses* 13.2.2.1 *and* 13.3)

Sl No. (1)	Detail Category (2)	Constructional Details: Illustration (*see* Note) (3)	Constructional Details: Description (4)
i)	118		**Rolled and extruded products** i) Plates and flats (1) ii) Rolled sections (2) iii) Seamless tubes (3) Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress.
ii)	103		**Bolted connections** (4) and (5): Stress range calculated on the gross section and on the net section. Unsupported one-sided cover plate connections shall be avoided or the effect of the eccentricity taken into account in calculating stresses **Material with gas-cut or sheared edges with no draglines** (6): All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress.
iii)	92		**Material with machine gas-cut edges with draglines or manual gas-cut material** (7) : Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress.

NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	92		**Welded plate I-section and box girders with continuous longitudinal welds** (8) & (9) : Zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having un-repaired stop-start positions.
ii)	83		**Welded plate I-section and box girders with continuous longitudinal welds** (10) & (11) : Zones of continuous automatic butt welds made from one side only with a continuous backing bar and all welds not having un-repaired stop-start positions. (12) : Zones of continuous longitudinal fillet or butt welds carried out from both sides but containing stop-start positions. For continuous manual longitudinal fillet or butt welds carried out from both sides, use Detail Category 92.
iii)	66		**Welded plate I-section and box girders with continuous longitudinal welds** (13) : Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions.
iv)	59		**Intermittent longitudinal welds** (14) : Zones of intermittent longitudinal welds
v)	52		**Intermittent longitudinal welds** (15) : Zones containing cope holes in longitudinally welded T-joints. Cope hole not to be filled with weld.
vi)	83		**Transverse butt welds (complete penetration)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (16) : Transverse splices in plates, flats and rolled sections having the weld reinforcement ground flush to plate surface. 100 percent NDT inspection, and weld surface to be free of exposed porosity in the weld metal. (17) : Plate girders welded as in (16) before assembly. (18) : Transverse splices as in (16) with reduced or tapered transition with taper ≤1:4

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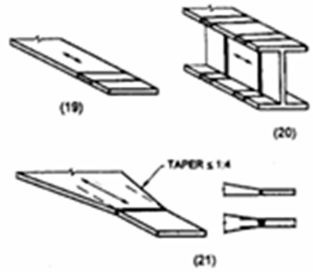
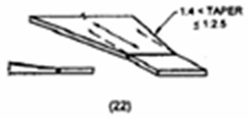
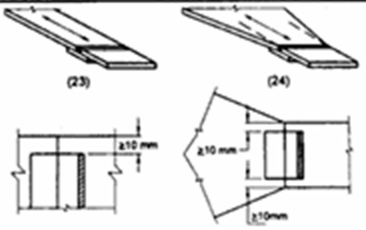
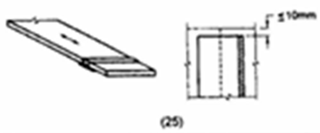
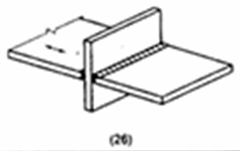
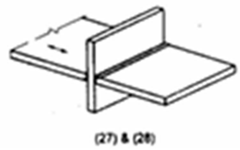
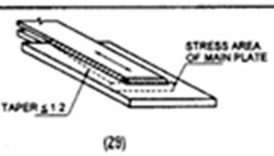
Sl No. (1)	Detail Category (2)	Constructional Details: Illustration (*see* Note) (3)	Constructional Details: Description (4)
vii)	66		**Transverse butt welds (complete penetration)** Welds run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (19) : Transverse splices of plates, rolled sections or plate girders. (20) : Transverse splice of rolled sections or welded plate girders, without cope hole. With cope hole use Detail Category 52, as in (15). (21) : Transverse splices in plates or flats being tapered in width or in thickness where the taper is ≤ 1:4.
viii)	59		**Transverse butt welds (complete penetration)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (22) : Transverse splices as in (21) with taper in width or thickness >1:4 but ≤1:2.5.
ix)	52		**Transverse butt welds (complete penetration)** (23) : Transverse butt-welded splices made on a backing bar. The end of the fillet weld of the backing strip shall stop short by more than 10 mm from the edges of the stressed plate. (24) : Transverse butt welds as per (23) with taper on width or thickness <1:2.5.
x)	37		**Transverse butt welds (complete penetration)** (25) : Transverse butt welds as in (23) where fillet welds end closer than 10 mm to plate edge.
xi)	52		**Cruciform joints with load-carrying welds** (26) : Full penetration welds with intermediate plate NDT inspected and free of defects. Maximum misalignment of plates either side of joint to be < 0.15 times the thickness of intermediate plate.
xii)	41 (27) 27 (28)		(27) : Partial penetration or fillet welds with stress range calculated on plate area. (28) : Partial penetration or fillet welds with stress range calculated on throat area of weld.
xiii)	46		**Overlapped welded joints** (29) : Fillet welded lap joint, with welds and overlapping elements having a design capacity greater than the main plate. Stress in the main plate to be calculated on the basis of area shown in the illustration.

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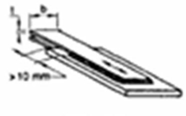
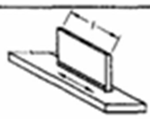
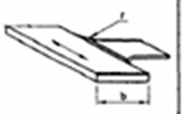
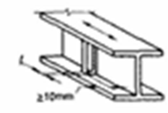
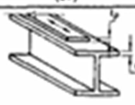
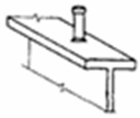
Sl No. (1)	Detail Category (2)			Constructional Details: Illustration (*see* Note) (3)	Constructional Details: Description (4)
xiv)	41	(30)		 (30) & (31)	**Overlapped weld joints** (30) : Fillet welded lap joint, with welds and main plate both having a design capacity greater than the overlapping elements.
xv)	33	(31)			(31) : Fillet welded lap joint, with main plate and overlapping elements both having a design capacity greater than the weld.
xvi)		(32)	(33)	 (32)  (33)	**Welded attachments (non-load carrying welds) — Longitudinal welds** (32) : Longitudinal fillet welds. Class of detail varies according to the length of the attachment weld as noted. (33) : Gusset welded to the edge of a plate or beam flange. Smooth transition radius (r), formed by machining or flame cutting plus grinding. Class of detail varies according to r/b ratio as noted.
	66	—	$1/3 \le r/b$		
	59	$l \le 50$ mm	—		
	52	$50 < l \le 100$ mm	$1/6 \le r/b < 1/3$		
	37	100 mm $< l$	—		
	33	—	$r/b < 1/6$		
xvii)	59			 (34)	**Welded attachments** (34) : Shear connectors on base material (failure in base material).
xviii)	59	$t \le 12$ mm		 (35)  (36)  (37)	**Transverse welds** (35) : Transverse fillet welds with the end of the weld ≥10 mm from the edge of the plate. (36) : Vertical stiffeners welded to a beam or plate girder flange or web by continuous or intermittent welds. In the case of webs carrying combined bending and shear design actions, the fatigue strength shall be determined using the stress range of the principal stresses. (37) : Diaphragms of box girders welded to the flange or web by continuous or intermittent welds.
	52	$t > 12$ mm			
xix)	37	t_f or t_p ≤25 mm		 (38)	**Cover plates in beams and plate girders** (38) : End zones of single or multiple welded cover plates, with or without a weld across the end. For a reinforcing plate wider than the flange, an end weld is essential.
	27	t_f or t_p >25 mm			
xx)	67			 (40)  (39)	**Welds loaded in shear** (39) : Fillet welds transmitting shear. Stress range to be calculated on weld throat area. (40) : Stud welded shear connectors (failure in the weld) loaded in shear (the shear stress range to be calculated on the nominal section of the stud).

NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

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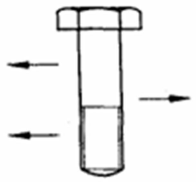
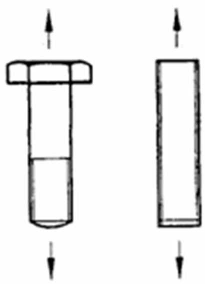
Table 26 (c) Detail Category Classification, Group 3 Bolts

(*Clauses* 13.2.2.1 *and* 13.3)

Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	83	 (41)	**Bolts in shear (8.8/TB bolting category only)** (41) : Shear stress range calculated on the minor diameter area of the bolt (A_c). NOTE — If the shear on the joint is insufficient to cause slip of the joint the shear in the bolt need not be considered in fatigue.
ii)	27	 (42)	**Bolts and threaded rods in tension (tensile stress to be calculated on the tensile stress area, A_t)** (42) : Additional forces due to prying effects shall be taken into account. For tensional bolts, the stress range depends on the connection geometry. NOTE — In connections with tensioned bolts, the change in the force in the bolts is often less than the applied force, but this effect is dependent on the geometry of the connection. It is not normally required that any allowance for fatigue be made in calculating the required number of bolts in such connections.

NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

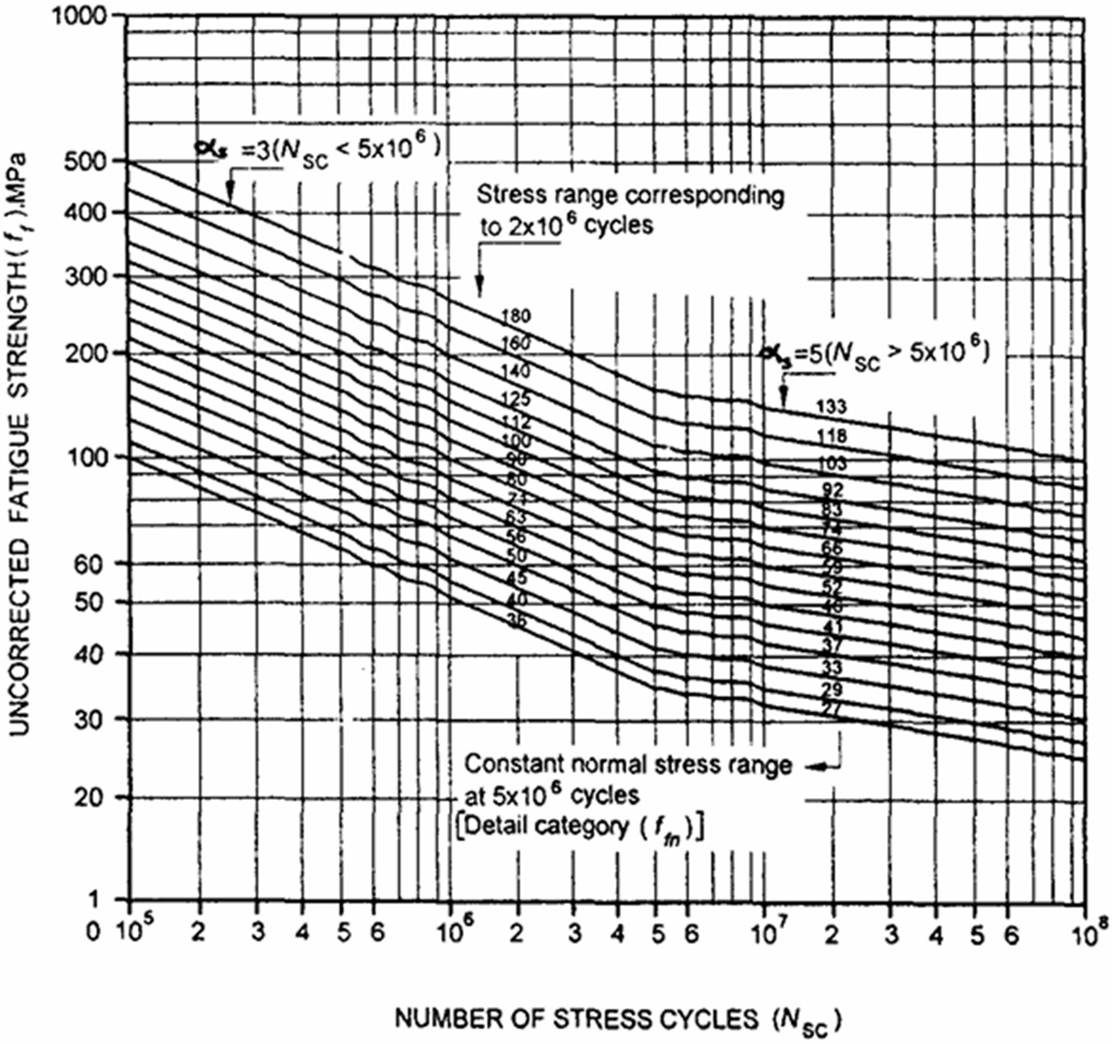
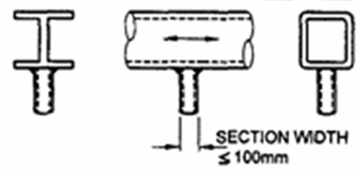
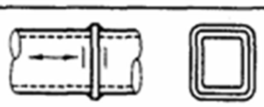


FIG. 22 *S-N* CURVE FOR NORMAL STRESS

Sl No. (1)	Detail Category (2)	Constructional Details: Illustration (*see* Note) (3)	Constructional Details: Description (4)
i)	103	 (43)	**Continuous automatic longitudinal welds** (43) : No stop-starts, or as manufactured, proven free to detachable discontinuities.
ii)	66 ($t \geq 8$ mm) 52 ($t < 8$ mm)	 (44)	**Transverse butt welds** (44) : Butt-welded end-to-end connection of circular hollow sections. NOTE — Height of the weld reinforcement less than 10 percent of weld with smooth transition to the plate surface. Welds made in flat position and proven free to detachable discontinuities.
iii)	52 ($t \geq 8$ mm) 41 ($t < 8$ mm)	 (45)	(45) : Butt-welded end-to-end connection of rectangular hollow sections
iv)	41 ($t \geq 8$ mm) 37 ($t < 8$ mm)	 (46)	**Butt welds to intermediate plate** (46) : Circular hollow sections, end-to-end butt-welded with an intermediate plate.
v)	37 ($t \geq 8$ mm) 30 ($t < 8$ mm)	 (47)	(47) Rectangular hollow sections, end-to-end butt welded with an intermediate plate
vi)	52	 (48)	**Welded attachments (non-load-carrying)** (48) : Circular or rectangular hollow section, fillet welded to another section. Section width parallel to stress direction ≤100 mm.
vii)	33 ($t < 8$ mm) 29 ($t < 8$ mm)	 (49)	**Fillet welds to intermediate plate** (49) : Circular hollow sections, end-to-end fillet welded with an intermediate plate.
viii)	29 ($t \geq 8$ mm) 27 ($t < 8$ mm)	 (50)	(50) : Rectangular hollow sections, end-to-end fillet welded with an intermediate plate.

NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be
4120 calculated on a plane normal to the arrow.

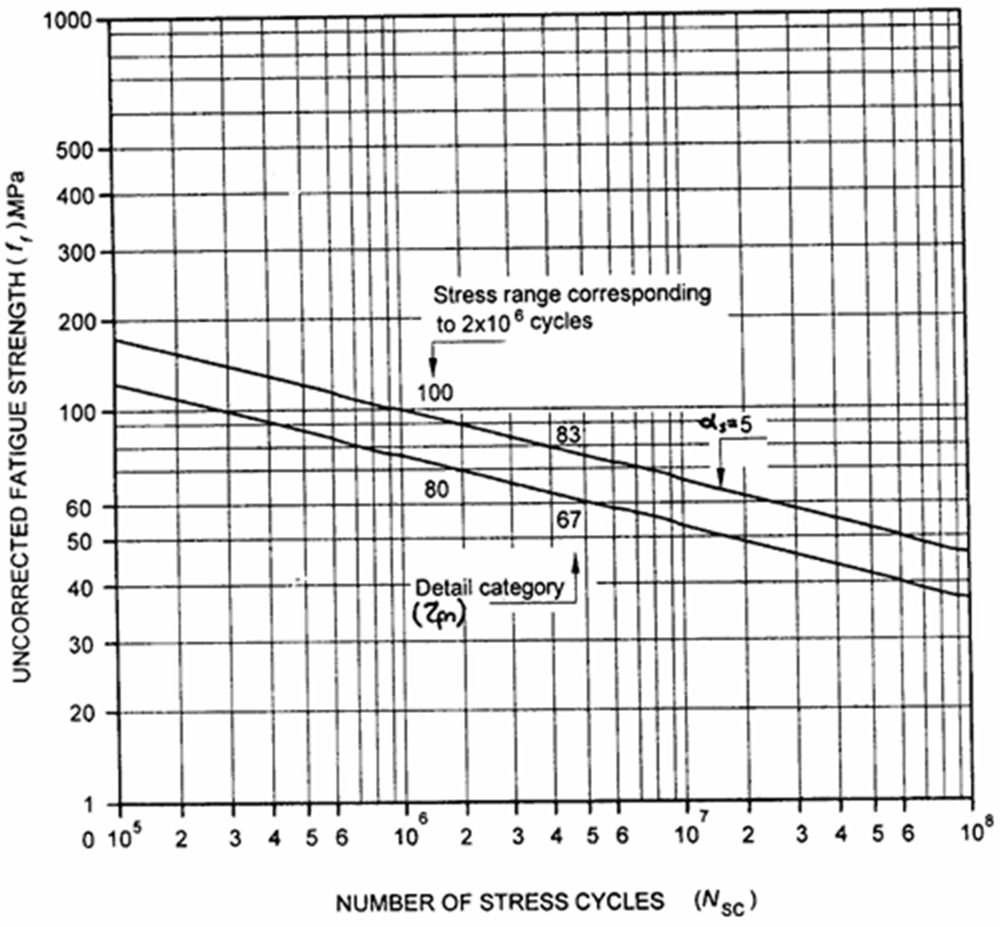


FIG. 23 *S-N* CURVE FOR SHEAR STRESS

13 Design Assisted by Testing

13.1.1 Need for Testing

Testing of structures, and members of components of structures is not required when designed in accordance with this standard. Testing may be accepted as an alternative to calculations or may become necessary in special circumstances.

Testing of a structural system, member of component may be required to assist the design in the following cases:

a) When the calculation methods available are not adequate for the design of a particular structure, member of component, testing shall be undertaken in place of design by calculation or to supplement the design by calculations.
b) Where rules or methods for design by calculation would lead to uneconomical design, experimental verification may be undertaken to avoid conservative design;
c) When the design of construction is not entirely in accordance with sections of this standard, experimental verification is recommended.
d) When confirmation is required on the consistency of production of material components, members or structures original designed by calculations or testing; and
e) When the actual performance of an existing structure capacity is in question, testing shall be used to confirm it.

13.1.2 Testing of structural system, member of component shall be of the following categories:

a) Proof testing- The application of test loads to a structure, sub-structure, member or connection to ascertain the structural characteristics of only that specific unit.
b) Prototype testing- Testing of structures, sub-structures, and members of connections is done to ascertain the structural characteristics of a class of such structures, sub-structures members of connection, which are nominally identical to the units tested.

13.2 Types of Tests

13.2.1 Acceptance test

This is intended as a non-destructive test for confirming structural performance. It should be recognized that the loading applied to certain structures might cause permanent distortions. Such effects do not necessarily indicate structural failure in the acceptance test. However, the possibility of their occurrence should be agreed to before testing.

The load for the acceptance test, $F_{test,a}$ shall be determined from:

$F_{test,a}$ = (Self weight) + ($1.15 \times$ Remainder of the permanent load) + (1.25 x Variable load).

The assembly shall satisfy the following criteria:

a) It shall demonstrate substantially linear behavior under test loading; and
b) On removal of the test load, the residual deflection shall not exceed 20 percent of the maximum-recorded deflection.

If the above criteria are not satisfied the test may be repeated one more time only, when the assembly satisfies the following criteria:

a) It shall demonstrate substantially linear behavior of the second application of test loading, and
b) Corresponding recorded residual deflection in the second test shall not exceed 10 percent of the maximum deflection during the test.

13.2.2 Strength Test

A strength test is used to confirm the calculated resistance of a structure or component. Where a number of items are to be constructed to a common design, and one or more prototypes are tested to confirm their strength, the others may be accepted without any additional tests, provided they are similar in all relevant respects of the prototype.

Before carrying out the strength test, the specimen should first be subjected to an satisfy acceptance test. Since the resistance of the assembly under test depends on the material properties, the actual yield strength of all the steel materials in the assembly shall be determined from coupon. The mean value of the yield strength, f_{ym} taken from such tests shall be determined with due regard to the importance of each element in the assembly. The strength test load $F_{test,s}$(including self-weight) shall be determined from:

$$F_{test,s} = \gamma_{mi} F_d \left(\frac{f_{ym}}{f_y}\right)$$

where, f_y =characteristics yield stress of material; F_d =factored design load for ultimate state

At this load, there shall be no failure by buckling or rupture of any part of the structure of component tests. On removal of the test load, the deflection should decrease by at least 20 percent of the maximum defection at $F_{test,s}$.

13.2.3 Test to Failure (Ultimate Strength Test)

The objective of a test to failure is to determine the design resistance from the ultimate resistance. In this situation, it is still desirable to carry out the acceptance and strength tests, before the test to failure.

Not less than three tests shall be carried out on nominally identical specimen. An estimate should be made of the anticipated ultimate resistance as a basis for such tests. During a test to failure, the loading shall first be applied in increments up to the strength test load. Subsequent load increments shall then be determined from consideration of the principal load deflection plot. The test load resistance, $F_{test,R}$ shall be determined as that load at which the specimen is unable to sustain any further increase in load. At this load, gross permanent distortion is likely to have occurred and, in some cases, such large gross deformation may define the test limit. If the deviation of any individual test result exceeds 10 percent of the mean value obtained for all three tests, at least three more tests shall be carried out. When the deviation from the mean does not exceed 10 percent of the mean, the design resistance may be evaluated as given below:

a) When the failure is ductile, the design resistance, F_d maybe determined from:

$$F_d = 0.9\, F_{test,Min} \left(\frac{f_y}{f_{ym}}\right) \gamma_{m0}$$

where, $F_{test,Min}$ = minimum test result from the tests to failure
f_{ym} = average yield strength from material tests;
f_y = characteristics yield stress of the grade of steel

b) In the case of a sudden (brittle) rupture type failure, the design resistance may be determined from:

$$F_d = 0.9\, F_{test,Min} \left(\frac{f_y}{f_{um}}\right) \gamma_{m1}$$

where, f_{um} = average ultimate strength from material tests
f_u = characteristics ultimate stress of the grade of steel

c) In the case of a sudden (brittle) buckling type failure, the design resistance shall be determined from:

$$F_d = 0.75\, F_{test,Min} \left(\frac{f_y}{f_{ym}}\right) \gamma_{m0}$$

d) In ductile buckling type failure in which the relevant slenderness γ can be reliably assessed the design resistance may be determined from:

$$F - d = 0.9 F_{test,Min} \left(\frac{\chi f_y}{\chi_m f_{ym}}\right) \gamma_{m0}$$

χ = reduction factor for the relevant buckling curve
χ_m = value of χ when the yield strength is f_{ym}

13.2.4 Check Tests

Where a component or assembly is designed on the basis of strength tests to failure and a production run is carried out for such items, an appropriate number of samples (not less than two) shall be selected from each production batch at random for check tests.

13.2.4.1 The samples shall be carefully examined to ensure that they are similar in all respects to the prototype testes, particular attention being given to the following:

a) Dimensions of components and connections
b) Tolerance and workmanship; and
c) Quality of steel used, checked with reference to mill tests certificates.

13.2.4.2 Where it is not possible to determine either the variations or the effect of variations from the prototype, an acceptance test shall be carried out as a check test.

13.2.4.3 In this check tests, the deflections shall be measured at the same positions as in the acceptance test of the prototype, the maximum measured deflection shall not exceed 120 percent of the deflection recorded during the acceptance tests on the prototype and the residual deflection should not be more than 105 percent of that recorded for the prototype.

13.3 Test Conditions

a) Loading and measuring devices shall be calibrated in advance.
b) The design of the test rig shall be such that:

i) The loading system adequately simulates the magnitude and distribution of the loading.
ii) It allows the specimen to perform in a manner representative of service conditions;
iii) Lateral and torsional restraint, if any, should be representative of those in service;
iv) Specimen should be free to deflect under load according to service donation
v) The loading system shall be able to follow the movements of the specimen without interruption or abnormal restraints; and
vi) In advertent eccentricities at the point application of the test loads and at the supports are avoided.

c) Test load shall be applied to the unit at a rate as uniform as practicable.
d) Deflections should be measured at sufficient points of high movements to ensure that the maximum value is determined.
e) If the magnitude of stresses in a specimen is to be determined, the strain at the desired location may be measured and the corresponding stress calculated.
f) Prior to any test, preliminary loading (not exceeding the characteristics values of the relevant loads) may be applied and then removed, in order to set the test specimen onto the test rig.

13.4 Test loading

13.4.1 Where the self-weight of the specimen is not representative of the actual permanent load in service, allowance for the difference shall be made in the calculation of test loads to be applied.

13.4.2 On the attainment of maximum load for either acceptance or strength tests, this load shall be maintained for at least 1 h. Reading of load and deflection shall be taken at intervals of 15 min and the loading shall be maintained constant until there is no significant increase in deflection during a 15 min period or until at least 1 h has elapsed.

13.4.3 The test load shall be equal to the design load for the relevant limit state in proof testing.

13.4.4 The test load in prototype testing shall be equal to the design load for the relevant limit state as multiplied by the appropriate factor given in Table 28.

Table 28 Factors to Allow for Variability of Structural Units

SN	No. of similar units to be tested	For strength limit state	For serviceability limit state
1	1	1.5	1.2
2	2	1.4	1.2
3	3	1.3	1.2
4	4	1.3	1.1
5	5	1.3	1.1
6	10	1.2	1.1

13.5 Criteria for Acceptance

13.5.1 Acceptance for Strength

The test structure, sub-structure, member of connection shall be deemed to comply with the requirement for strength if it's able to sustain the strength test load for at least 15 min.

It shall then be inspected to determine the nature and extent of any damage incurred to ruin the test. The effects of the damage shall be considered and if necessary appropriate repairs to the damaged parts carried out.

13.5.2 Acceptance of Serviceability

The maximum deformation of the structure or member under the serviceability limit state test load shall be within the serviceability limit values appropriate to the structure.

14 Durability

14.1 General

A durable steel structure is one that performs satisfactorily the desired function in the working environment under the anticipated exposure condition during its service life, without deterioration of the cross-sectional area and loss of strength due to corrosion. The materials used, the detailing, fabrication, erection and surface protection measures should all address the corrosion protection and durability requirements.

14.2 Requirements of Durability

14.2.1 Shape, size, orientation of members, connections and details

The design, fabrication and erection details of exposed structures should be such that good drainage of water is ensured. Standing pool of water, moisture accumulation and rundown of water for extended duration shall be avoided.

The details of connections should ensure that:

a) All exposed surfaces are easily accessible for inspection and maintenance; and
b) All surfaces, not so easily accessible are completely sealed against ingress of moisture.

14.2.2 Exposure Condition

14.2.2.1 General environment

The general environment, to which a steel structure is exposed during its working life, is classified into five levels of severity, as given in Table 29.

Table 29: Environmental Exposure Conditions

SN	Environmental Classification	Exposure Conditions
i)	Mild	Surface normally protected against exposure to weather or aggressive condition as in interior of buildings
ii)	Moderate	Structural steel surfaces: a. exposed to condensation and rain b. continuously under water c. exposed to non-aggressive soil or groundwater
iii)	Severe	Structural steel surfaces: a. exposed to severe frequent rain b. exposed to alternate wetting and drying c. severe condensation
iv)	Very Severe	Structural steel surface exposed to: a. corrosive fumes b. aggressive sub soil or ground water
v)	Extreme	Structural steel surfaces exposed to aggressive liquid or solid chemicals

14.2.2.2 Abrasion

Specialist literature may be refereed for durability of surfaces exposed to abrasive action as in machinery, conveyor belt support system, storage bins for grains or aggregates.

14.2.2.3 Exposure to sulphate attack

Appropriate coating may be used when surfaces of structural steel are exposed to concentration of sulphates (SO_3) in soil, ground water, etc.

When exposed to very high sulphate concentrations of more than 2 percent in soil and 5 percent in water, some form of lining such as polyethylene, polychloroprene sheet or surface coating based on asphalt, chlorinated rubber, epoxy or polymethine material should be used to completely avoid access of the solution to the steel surface.

14.2.3 Corrosion Protection Methods

The methods of corrosion protection are government by actual environmental conditions as specified in IS 9077 and IS 9172. The main corrosion protection methods are: a) Controlling the electrode potential; b) Inhibitors, and c) Inorganic/metal coatings or organic/paint systems.

14.2.4 Surface Protection

14.2.4.1 In the case of mild exposure, a coat of primer after removal of any loose mill scale may be adequate. As the exposure condition becomes more critical, more elaborate surface preparations and coatings become necessary. In case of extreme environmental classification protection shall be as per specialist literature. Table 30, Table 31 and Table 32 gives guidance to protection of steelwork for different desired lives.

Table 30: Protection Guide for Steel Work Application: Desired Life of Coating System in Different Environments

SN	Atmospheric Condition/ Environmental Classification	Coating System					
		1	2	3	4	5	6
i)	Normal inland (rural and urban areas), mild	12 years	18 years	20 years	About 20 years	About 20 years	About 20 years
ii)	Polluted inland (high airborne sulphur dioxide), moderate	10 years	15 years	12 years	About 18 years	15-20 years	Above 20 years
iii)	Severe	10 years	12 years	20 years	About 20 years	About 20 years	Above 20 years
iv)	Very severe or extreme	8 years	10 years	10 years	About 15 years	15-20 years	Above 20 years

Table 31: Protection Guide for Steel Work Application: Specification for Different Coating System (Shop Applied Treatments)

SN	Protection	Coating System					
		1	2	3	4	5	6
i)	Surface preparation	Blast clean	Blast clean	Blast clean	Blast clean	Girt Blast	Blast clean
ii)	Pre-fabrication primer	Zinc phosphate epoxy, 20 μm	2 pack zinc-rich epoxy,20 μm	-	2 pack zinc-rich epoxy, 20 μm	-	Ethyl zinc silicate, 20 μm
iii)	Post-fabrication primer	High-build zinc phosphate modified alkyd, 60 μm	2 pack zinc-rich epoxy, 20 μm	Hot dip epoxy, galvanized, 85 μm	2 pack zinc-rich epoxy, 25 μm	Sprayed zinc or sprayed aluminum	Ethyl zinc silicate, 60 μm
iv)	Intermediate coat	-	High-build zinc phosphate, 25 μm	-	2 pack epoxy micaceous iron oxide	Sealer	Chlorinated rubber alkyd 35 μm
v)	Top coat	-	Modified Alkyd Micaceous iron oxide, 50 μm	-	2 pack epoxy micaceous iron oxide,85 μm	Sealer	-

Table 32: Protection Guide for Steel Work Application — Specification for Different Coating System (Site Applied Treatments)

SN	Protection	Coating System					
		1	2	3	4	5	6
i)	Surface preparation	As necessary	As necessary	No site treatment	As necessary	No site treatment	As necessary
ii)	Primer	Touch in	Touch in	-	-	-	Touch in
iii)	Intermediate coat	-	Modified Alkyd Micaceous iron oxide, 50 μm	-	Touch in	-	High-build micaceousir on oxide Chlorinated rubber Micaceous, 75 μm
iv)	Top coat	High-build Alkyd finish, 60 μm	Modified Alkyd Micaceous iron oxide, 50 μm	-	High-build chlorinated rubber	-	High-build iron oxide Chlorinated rubber, 75 μm

14.2.4.2 Steel surface shall be provided with at least one coat of primer immediately after its surface preparation such as by sand blasting to remove all mill scale and rust to expose the steel.

14.2.4.3 Steel without protective coating shall not be stored for long duration in outdoor environment.

14.2.4.4 Surface to transfer forces by friction as in HSFG connections shall not be painted. However it shall be ensured that moisture is not trapped on such surfaces after pre-tensioning of bolts by proper protective measures.

14.2.4.5 Members to be assembled by welding shall not be pre-painted at regions adjacent to the location of such welds. However, after welding, appropriate protective coatings shall be applied in the region, as required by the exposure conditions. If the contact surfaces cannot be properly protected against ingress of moisture by surface coating, they may be completely sealed by appropriate welds.

14.2.4.6 Pre-painted members shall be protected against abrasion of the coating during transportation, handling and erection.

14.2.5 Special Steels

Steels with special alloying elements and production process to obtained better corrosion resistance may be used as per specialist literature.

15 Fire Resistance

15.1 Requirements

The section applies to steel building elements designed to have a required fire-resistance (FRL) as per the relevant specifications.

15.1.1 For protected steel members and connections, the thickness of protection material (h_i) shall be greater than or equal to that needed to give a period of structural adequacy (PSA) greater than or equal to the required FRL.

15.1.2 For unprotected steel members and connections, the exposed surface area to mass ratio (k_{sm}) shall be less than or equal to that required to give a PSA equal to the required FRL.

15.2 Fire Resistance Level(FRL)

The required FRL shall be as prescribed in building specifications or as required by the user or the municipality ordinance. The FRL specified in terms of the duration (in minutes) of standard fire load without collapse depends upon: a) the purpose for which structure is used, and b) the time taken to evacuate in case of fire.

15.3 Period of Structural Adequacy (PSA)

15.3.1 The calculation of PSA involves:

a) Calculation of the strength of the element as a function of temperature of the element and the determination of limiting temperature;
b) Calculation of the thermal response of the element, that is calculation of the variation of the temperature of the element or the parts of the element with time, when exposed to fire; and
c) Determination of PSA at which the temperature of the element or parts of the element reaches the limiting temperature.

15.3.2 Determination of Period of Structural Adequacy

The period of structural adequacy (PSA) shall be determined using one of the following methods:

a) By calculation:

i) By determining the limiting temperature of the steel (T_l) in accordance with 15.5; and
ii) By determining the PSA as the time (in min) from the start of the test to the time at which the limiting steel temperature (t) is attained, in accordance with 15.6 for protected members and 15.7 for unprotected members; or

b) By direct application of a single test in accordance with 15.8; or
c) By calculation of the temperature of the steel member by using a rational method of the analysis confirmed by test data or by methods available in special literature.

15.4 Variation of mechanical properties of steel with temperature

15.4.1 Variation of Yield Stress with Temperature:

The influence of temperature on the yield stress of steel shall be taken as follows:

$$\frac{f_y(T)}{f_y(20)} = 1\text{, when } 0\,°C < T \leq 215\,°C\text{; and}$$

$$= \frac{905-T}{690}\text{, when } 215\,°C < T \leq 905\,°C\text{; and}$$

where, $f_y(T) =$ Yield stress of steel at $T°\,C$; $T =$ Temperature of the steel in $°C$

$f_y(20) =$ Yield stress of steel at $20°C$ (Room temperature) and

This relationship is shown by Curve 1 in Figure 28.

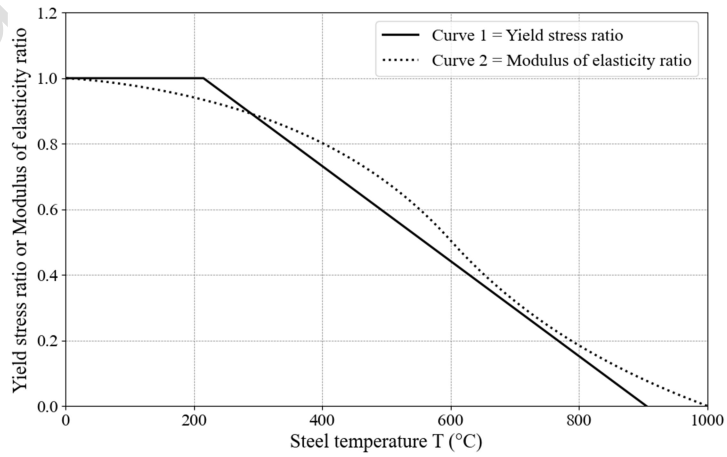


Figure 28: Variation of Mechanical Properties of steel with temperature

15.4.2 Variation of Modulus of Elasticity with Temperature

The influence of temperature on the modulus of elasticity shall be taken as follows:

$$\frac{E(T)}{E(20)} = 1.0 + \left[\frac{T}{2000\left[\mathrm{Ln}\left(\frac{T}{1100}\right)\right]}\right],\ \text{when } 0°C < T \leq 600\ °C\ ;\ \text{and}$$

$$= \frac{690\left(1-\frac{T}{1000}\right)}{T-53.5},\ \text{when } 600°C < T \leq 1000\ °C\ ;$$

where, $E(T) =$ Modulus of elasticity of steel at $T°C$; $E(20) =$ Modulus of elasticity of steel at $20°C$

The relationship is shown by Curve 2 in Figure 28.

15.4.3 For special steel with higher temperature resistance, the manufacturer's recommendation shall be used to obtain the variation of f_y and E.

15.5 Limiting steel temperature

The limiting steel temperature T_l shall be calculated as follows:

$$T_l = 905 - 690\, r_f$$

where, $r_f =$ ratio of the design action on the member under fire to the design capacity of the member ($R_d = R_u/\gamma_m$) at room temperatures,
$R_d, R_u =$ design strength and ultimate strength of the member at room temperature respectively, and
γ_m= partial safety factor for strength

The design action under fire shall consider the following: a) Reduced bond likely under fire, and b) Effects of restraints to expansion of the element during fires.

Limiting steel temperature for special steels may be appropriately calculates using the thermal characteristics of the material obtained from the supplier of the steel.

15.6 Temperature Increase with Time in Protected Members

15.6.1 The time (t) at which the limiting temperature (T_L) is attained shall be determined from the results of a single test in accordance with 15.6.2, or by calculation on the basis of a suitable series of fire tests in accordance with 15.6.3.

15.6.1.1 For beams and for all members with a four-sided fire exposure condition, the limiting temperature(T_L) shall be taken as the average of all of the temperature measured at the thermocouple locations shown in the standard fire test method

15.6.1.2 For columns with a three-sided fire exposure condition, the limiting temperature (T_L) shall be taken as the average of the temperatures measured at the thermocouple locations on the face farthest from the wall. Alternatively, the temperature from members with a four-sided fire exposures condition and the same section factor may be used.

15.6.2 Temperature Based on a Test Series

Calculation of the variation of steel temperature with time shall be by interpolation of the results of series of fire tests using equation 15.6.2.1**,** subject to the limitations and conditions of 15.6.2.2.

15.6.2.1 Regression analysis

The relationship between temperature (T) and time (t) for a series of tests on a group shall be calculated by least-squares regression as follows:

$$t = k_o + k_1 h_i + k_2\left(\frac{h_i}{k_{sm}}\right) + k_3 T + k_4 h_i T + k_5\left(\frac{h_i T}{k_{sm}}\right) + k_6\left(\frac{T}{k_{sm}}\right)$$

Where, t=time from the start of the test, in minutes
k_0 to k_6= regression coefficients, determined for use in equation
h_i= thickness of fire protection materials, in millimeters
T= steel temperature, in degree Celsius, $T > 250\,°C$
k_{sm}=exposed surface area to mass ratio, in square meters/tonne)

In lieu of test results, the values for coefficients in Table 33 may be used in the equation 15.6.2.1 when the test satisfies the conditions specified in 15.6.2.2.

Table 33: Regression Coefficients, k

K_0	K_1	K_2	K_3	K_4	K_5	K_6
-25.9	1.698	-13.71	0.03	0.0005	0.5144	6.633

15.6.2.2 Limitations and conditions on use of regression analysis

Test data to be utilized in accordance with 15.6.2.1 shall satisfy the following:

a) Steel members shall be protected with board, sprayed, blanket or similar insulation materials having a dry density less than 1000 kg/m^3,
Note: Intumescent coatings do not fulfil this criterion and hence do not come within the scope of this code. They may be tested and assessed in accordance with suitable literatures.
b) All tests shall incorporate the same fire protection system;
c) All members shall have the same fire exposure condition;
d) The test series shall include at least 9 tested members;
e) The test series may include prototypes which have not been loaded provided that stickability has been demonstrated;
f) All members subject to a three-sided fire exposure condition shall be within a group in accordance with 15.9.

The regression equation obtained for one tire protection system may be applied to another system using the same fire protection material and the same fire exposure condition provided that stack ability has been demonstrated for the second system.

A regression equation obtained using prototypes with a four-sided fire exposure condition maybe applied to a member with a three-sided fire exposure condition provided that stack ability has been demonstrated for the three-sided case.

15.6.3 Temperature Based on Single Test

The variation of steel temperature with time measured in a standard fire test maybe used without modification provided:

a) Fire protection system is the same as the prototype;
b) Fire exposure condition is the same as the prototype;
c) Fire protection material thickness is equal to or greater than that of the prototype;
d) Surface area to mass ratio is equal to or less than that of the prototype; and
e) Where the prototype has been submitted to a standard fire test in an unloaded condition, stickability has been separately demonstrated.

15.6.4 Parameters of Importance in the Standard Fire Test

a) Specimen type, loading, configuration;
b) Exposed surface area to mass ratio;
c) Insulation type, thermal properties d) thickness; and
d) Moisture content of the insulation material.

15.7 Temperature increase with time in unprotected members

The time (t) at which the limiting temperature (T_L) is attained shall be calculated for:

a) Three-sided exposure as follows:

$$t = -5.2 + 0.0221\, T_L + \frac{0.433T_L}{k_{sm}}$$

b) Four-sided exposure as follows:

$$t = -4.7 + 0.0263\, T_L + \frac{0.213T_L}{k_{sm}}$$

where, $t =$ time from the start of the test, in min.
$T =$ steel temperature, in $°C, 500\,°C \leq T \leq 750\,°C$, and
$k_{sm} =$ exposed surface area to mass ratio, $2 \times 10^3 \frac{mm^2}{kg} \leq k_{sm} \leq 35 \times 10^3 \frac{mm^2}{kg}$.

For temperatures below 500$°C$, linear interpolation shall be used, based on the time at 500$°C$ and an initial temperature of 20$°C$ at t equals 0.

15.8 Determination of PSA from a single Test

The period of structural adequacy (PSA) determined from a single test maybe applied without modification provided:

a) Conditions, specified in 15.6.3 are satisfied
b) Conditions of support are the same as the prototype and the restraints are not less favorable than those of the prototype, and
c) Ratio of the design load for fire to the design capacity of the member is less than or equal to that of the prototype.

15.9 Three-sided Fire Exposure Condition

Members subject to a three-sided fire exposure condition shall be considered in separate groups unless the following conditions are satisfied:

a) The characteristics of the members of a group as given below, shall not vary from one another by more than
   i. Concrete density: $\frac{highest\ in\ group}{lowest\ in\ group} \leq 1.25$, and
   ii. Effective thickness(h_e): $\frac{largest\ in\ group}{smallest\ in\ group} \leq 1.25$, and
   iii. Where the effective thickness (h_c) is equal to the cross-sectional area excluding voids per unit width, as shown in Figure 30.
b) Rib voids shall either be:
   a) All open; or
   b) All blocked as shown in Figure 30.
c) Concrete slabs may incorporate permanent steel deck formwork

15.10 Special Considerations

15.10.1 Connections

Connections shall be protected with the maximum thickness of fire protection material required for any of the members framing into the connection to achieve their respective fire-resistance levels. This thickness shall be maintained over all connection components, including bolt heads, welds and splice plates.

15.10.2 Web Penetrations

The thickness of tire protection material at and adjacent to web penetrations shall be the greatest of that required, when:

a) area above the penetration is considered as a three-sided fire exposure condition (k_{sm}) (Figure 30).

b) area below the penetration is considered as a four-sided fire exposure condition (k_{sm}) (Figure 30).and
c) section as a whole is considered as a three- Rating sided fire exposure condition (k_{sm}) (Figure 30).

This thickness shall be applied over the full beam depth and shall extend on each side of penetration for a distance at least equal to the beam depth and not less than 300 mm.

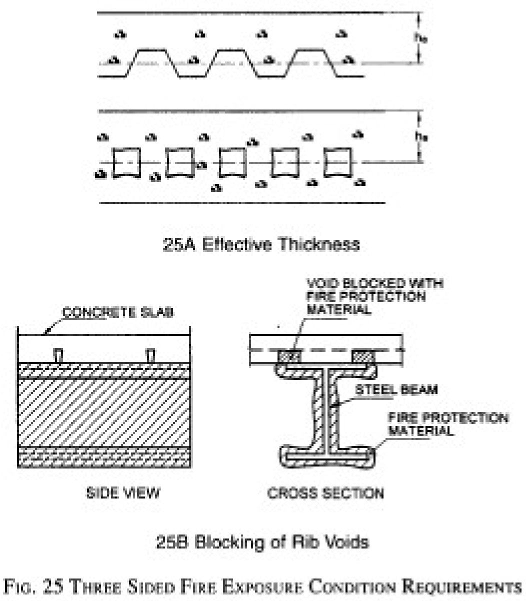


Figure 29: Three Sided Fire Exposure Condition Requirements

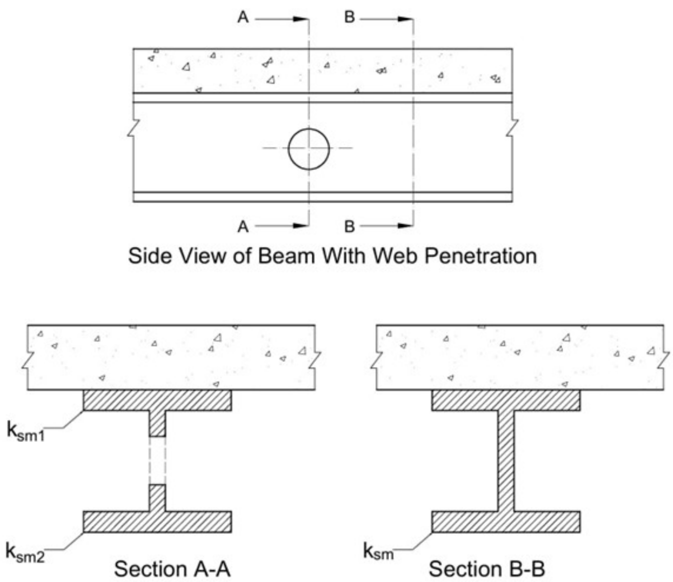


Figure 30: Web Penetration

15.11 Fire Resistance Rating

The fire resistance rating of various building components such as walls, columns, beams, and floors are given in Table 31 and Table 32. Fire damage assessment of various structural elements of the building and adequacy of the structural repairs can be done by the fire resistance rating for encased steel column and beam.

Table 34: Encased Steel Columns, 203 mm X 203 mm (Protection Applied on Four Sides)

SN	Nature of Construction and Materials	Minimum Dimensions Excluding Any Finish, for a Fire Resistance of (mm)				
		1 h	1.5 h	2 h	3 h	4 h
i)	Hollow protection (without an air cavity over the flanges):					
a)	Metal lathing with troweled lightweight aggregate gypsum plaster [1]	13	15	20	32	-
b)	Plasterboard with 1.6 mm wire binding at 100 mm pitch, finished with lightweight aggregate gypsum plaster less than the thickness specified:					
	1) 9.5 mm plaster board	10	15	-	-	-
	2) 19 mm plaster board	10	13	20	-	-
c)	Asbestos insulating boards, thickness of board:					
	1) Single thickness of board, with 6 mm cover fillets at transverse joints	-	19	25	-	-
	2) Two layers, of total thickness	-	-	-	38	50
d)	Solid bricks of clay, composition or sand lime, reinforced in every horizontal joint, un-plastered	50	50	50	75	100
e)	Aerated concrete blocks	60	60	60	-	-
f)	Solid blocks of lightweight concrete bellow protection (with an air cavity over the flange)	50	50	50	60	75
ii)	Asbestos insulating board screwed to 25 mm asbestos battens	12	19	-	-	-
iii)	Solid protections					
a)	Concrete, not leaner than 1:2:4 mix (un-plastered):					
	1) Concrete not assumed to be load bearing, reinforced [2]	25	25	25	50	75
	2) Concrete assumed to be load bearing	50	50	50	75	75
b)	Lightweight concrete, not leaner than 1:2:4 mix (un-plastered) concrete not assumed to be load bearing, reinforced [2]	25	25	25	25	25

[1] So fixed or designed, as to allow full Penetration for mechanical bond.

[2] Reinforcement shall consist of steel blinding wire not less than 2.3 mm diameter, or a steel mesh weighing not less than 0.5 $\frac{Kg}{m^3}$.In concrete protections, the spacing of the reinforcement shall not exceed 200 mm in any direction.

Table 35: Encased Steel Columns, 203 mm X 203 mm (Protection Applied on Three Sides)

SN	Nature of Construction and Materials	Minimum Dimensions Excluding Any Finish, for a Fire Resistance of (mm)					
		0.5 h	1 h	1.5 h	2 h	3 h	4 h
i)	Hollow protection (without an air cavity beneath the lower flanges):						
a)	Metal lathing with troweled lightweight aggregate gypsum plaster [1]	13	13	15	20	25	-
b)	Plasterboard with 1.6 mm wire binding at 100 mm pitch, finished with lightweight aggregate gypsum plaster less than the thickness specified: [2]						
	1) 9.5 mm plaster board	10	10	15	-	-	-
	2) 19 mm plaster board	10	10	13	20	-	-
c)	Asbestos insulating boards, thickness of board:						
	1) Single thickness of board, with 6 mm cover fillets at transverse joints	-	-	19	25	-	-
	2) Two layers, of total thickness	-	-	-	-	38	50
ii)	Hollow protection (with an air cavity below the lower flange)						
a)	Asbestos insulating board screwed to 25 mm asbestos battens	9	12	-	-	-	-

iii)	Solid protections						
a)	Concrete, not leaner than 1:2:4 mix (un-plastered):						
	1) Concrete not assumed to be load bearing, reinforced [3]	25	25	25	25	50	75
	2) Concrete assumed to be load bearing	50	50	50	50	75	75
b)	Lightweight concrete, not leaner than 1:2:4 mix (un-plastered) concrete not assumed to be load bearing, reinforced [4]	25	25	25	25	40	60

[1] *So fixed or designed, as to allow full penetration for mechanical bond.*

[2] *Where wire binding cannot be used, expert advice should be sought regarding alternative methods of support to enable the lower edges of the plaster board to be fixed together and to the lower flange, and for the top edge of the plasterboard to be held in position*

[3] *Reinforcement shall consist of steel binding wire not less than 2.3 mm in diameter, or a steel mesh weighing not less than 0.5 kg/m^2. In concrete protection, the spacing of the reinforcement shall not exceed 200 mm in any direction.*

[4] *Concrete not assumed to be load bearing, reinforced.*

16 Fabrication and Erection

16.1 General

Tolerances for fabrication of steel structures shall conform to IS 7215. Tolerances for erection of steel structures shall conform to IS 12483. For general guidance on fabrication by welding, reference may be made to IS 9595.

16.2 Fabrication Procedures

16.2.1 Straightening

Material shall be straightened or formed to the specified configuration by methods that will not reduce the properties of the material below the values used in design. Local application of pressure at room or at elevated temperature or other thermal means may be used for straightening, provided the above is satisfied.

16.2.2 Clearances

The erection clearance for cleated ends of members, connecting steel to steel should preferably be not greater than 2.0 mm at each end. The erection clearance at ends of beams without cleats should be not more than 3 mm at each end. Where for practical reasons, greater clearance is necessary, suitably designed seating should be provided.

16.2.2.1 The holes for bolts may be made as given in Table 20 unless otherwise specified by engineer. The hole diameter in base plates shall not exceed the anchor bolt diameter by more than 6 mm.

16.2.2.2 In friction type of connection clearance may be maintained, unless specified otherwise in the design document.

16.2.3 Cutting

Cutting shall be affected by sawing, shearing, cropping, machining or thermal cutting process. Shearing, cropping and gas cutting shall be clean, reasonably square, and free from any distortion. Should the inspector find it necessary, the edges shall be ground after cutting. Planning or finishing of sheared or gas-cut edges of plates or shapes shall not be required, unless specially noted on drawing or included in stipulated edge preparation for welding or when specifically required in the following section.

Re-entrant corners shall be free from notches and have largest practical radii with a minimum radius of 15 mm.

16.2.3.1 Shearing

Shearing of items over 16 mm thick to be galvanized and subject to tensile force or bending moment shall not be carried out, unless the item is stress relieved subsequently.

The use of sheared edges in the tension area shall be avoided in location subject to plastic hinge rotation at factored loading.

16.2.3.1.1 Thermal cutting

Gas cutting of high tensile steel by mechanically controlled torch may be permitted, provided special care is taken to leave sufficient metal to be removed by machining, so that all metal that has been hardened by flame is removed. Hand flame cutting may be permitted only subject to the approval of the inspecting authority.

Except where the material is subsequently joined by welding, no load shall be transmitted through a gas cut surface.

Thermally cut free edges, which shall be subject to calculated static tensile stress shall be free from round bottom gouges greater than 5 mm deep. Gouges greater than 5 mm deep and notches shall be removed by grinding.

16.2.4 Holing

16.2.4.1.1 Holes though more than one thickness of material for members, such as compound stanchion and girder flanges, shall be where possible, drilled after the members are assembled and tightly clamped or bolted together. Around hole for a bolt shall either be machine flame cut, or drilled full size, or sub-punched 3 mm undersize and reamed to size or punched full size.

Hand flame cutting of a bolt hole shall not be permitted except as a site rectification measure for holes in column base plates.

16.2.5 Punching

A punched hole shall be permitted only in material whose yield stress (f_y) does not exceed 360 MPa and where thickness does not exceed $(5600/f_y)$ mm. In cyclically loaded details, punching shall be avoided in plates with thickness greater than 12mm. For greater thickness and cyclically loaded details, holes shall be either drilled from the solid or sub-punched or sub-drilled and reamed. The die for all sub-punched holes or the drill for all sub-drilled holes shall be at least 3 mm smaller than the required diameter of finished hole.

16.2.6 Oversize holes

A special plate washer of minimum thickness 4 mm shall be used under the nut, if the hole diameters is larger than the bolt diameter by 3 mm or more.

Oversize hole shall not exceed $1.25d$ or $(d + 8)$ mm in diameter, where d is the nominal bolt diameter, in mm.

A short slotted hole shall not exceed the appropriate hole size in width and $1.33d$ in length. A long slotted hole shall not exceed the appropriate hole size in width and $2.5d$ in length. If the slot length is larger than those specified, shear transfer in the direction of slot is not admissible even in friction type of connection.

Slotted holes shall be punched either in one operation or else formed by punching or drilling two round holes apart and completed by high quality mechanically controlled flame cutting and dressing to ensure that bolt can freely travel the full length of the slot.

16.2.6.1 Fitted bolt holes

Holes for turned and fitted bolts shall be drilled to a diameter equal to the nominal diameter of the shank or barrel subject to tolerance specified in IS 919 (parts 1 and 2). Preferably, parts to be connected with close tolerance of barrel bolts shall be firmly held together by tacking bolts or clamps and the holes drilled through all the thickness at one operation and subsequently reamed to size. All holes not drilled through all thickness at one operation shall be drilled to a smaller size and reamed out after assembly. Where this not practicable, the parts shall be drilled and reamed separately through hard bushed steel jigs.

16.2.6.1.1 Holes for bolts shall not be formed generally by gas cutting process. However, advanced gas cutting processes such as plasma cutting may be used to make holes in statically loaded members only. In cyclically loaded members subjected to tensile stresses which are vulnerable under fatigue, gas cutting shall not be used unless subsequent reaming is done to remove the material in the heat affected zone around the hole.

16.3 Assembly

All parts of bolted members shall be pinned or bolted and rigidly held together during assembly.

The component parts shall be assembled and aligned in such a manner that they are neither twisted nor otherwise damaged, and shall be so prepared that the specified camber, if any, is provided.

16.3.1 Holes in assembly

When holes are drilled in one operation through two or more separable parts, these parts, when so specified by the engineer, shall be separated after drilling and the burrs removed.

Matching holes for black bolts shall register with each other so that a gauge of 1.5 mm or 2.0 mm (as the case may be, depending on whether the diameter of the bolt is less than or more than 25 mm) less in diameter of the hole will pass freely through the assembled members in the direction at right angle to such members.

Drilling done during assembly to align holes shall not distort the metal or enlarge the holes.

Holes in adjacent part shall match sufficiently well to permit easy entry of bolts. If necessary, holes except oversize or slotted holes may be enlarged to admit bolts, by moderate amount of reaming.

16.3.2 Thread length

When the design is based on bolts with unthreaded shanks in the shear plane, appropriate measures shall be specified to ensure that, after allowing for tolerance, neither the threads nor the thread run-out be in the shear plane. The length of bolt shall be such that at least one clear thread shows above the nut and at least one thread plus the thread run out is clear beneath the nut after tightening. One washer shall be provided under the rotated part.

16.3.3 Assembly subjected to vibration

When non-preloaded bolts are used in a structure subject to vibration, the nuts shall be secured by locking devises or other mechanical means. The nuts of preloaded bolts may be assumed to be sufficiently secured by the normal tightening procedure.

16.3.4 Washers

Washers are not normally required on non-preloaded bolts, unless specified otherwise. Tapered washers shall be sued where the surface is inclined at more than 3° to a plane perpendicular to the bolt axis. Hardened washer shall be used for preloaded bolts or the nut, whichever is to be rotated. All material within the grip of the bolt shall be steel and no compressible material shall be permitted in the grip.

16.4 Bolting

16.4.1 In all cases where the full bearing area of the bolt is to be developed, the bolt shall be provided with a washer of sufficient thickness under the nut to avoid any threaded portion of the bolt being within the thickness or the parts bolted together, unless accounted for in design.

16.4.2 Pre-tensioned bolts shall be subjected to initial tension (the proof stress) by an appropriate pre-calibrated method.

16.5 Welding

16.5.1 Welding shall be in accordance with IS 816, IS 819, IS 1024, IS 1261, IS 1323 and IS 9595, as appropriate.

16.5.2 For welding of any particular type of joint, welders shall give evidence acceptable to purchaser of having satisfactorily completed appropriate tests as prescribed in IS 817, IS 1393, IS 7307 (Part 1), IS 7310 (part 1) and IS 7318 (Part 1), as relevant.

16.5.3 Assembly and welding shall be carried out in such a way to minimize distortion and residual stress and that the final dimensions are within appropriate tolerances.

16.6 Machining of Buts, Caps and Bases

16.6.1 Column splices and butt joints of struts and compression members, depending on contact for stress transmission, shall be accurately machined and close-butted over the whole section with a clearance not exceeding 0.2 mm locally, at any place. Sum of all such clearance shall not be more than 30 percent of the contact area for stress transmission. In column caps and bases, the ends of shafts together with the attached gussets, angles, channels, etc; after connecting together should be accurately machine so that clearance between the contact surfaces shall not exceed 2 mm locally, subject further to the condition that sum total of all such clearance shall not exceed 30 percent of the total contact area for stress transmission. Care should be taken that these gussets, connecting angles or channels are fixed with such accuracy that they are not reduced in thickness by machining by more than 2.0 mm.

16.6.2 Where sufficient gussets and welds are provided to transmit the entire loading (see section 4) the column ends need not be machined.

16.6.3 Slab Bases and Caps

Slab bases and slab caps, except when cut from material with true surfaces, shall be accurately machined over the bearing surfaces and shall be in effective contact with the end of the stanchion, the bearing face which is to be grouted to fit tightly at both top and bottom, unless welds are provided to transmit the entire column face.

16.6.4 To facilitate grouting, sufficient gaps shall be left between the base plates and top of pedestal and holes shall be provided where necessary in stanchion bases for the escape of air.

16.7 Painting

16.7.1 Painting shall be done in accordance with IS 1477 (Parts 1 and 2).

16.7.2 All surfaces, which are to be painted, oiled or otherwise treated, shall be dry and thoroughly cleaned to remove all loose scale and loose rust.

16.7.3 Shop contact surfaces need not be painted unless specified. If so specified, they shall be brought together while the paint is still wet.

16.7.4 Surfaces not in contact, but inaccessible after shop assembly, shall receive the full specified protective treatment before assembly. This does not apply to the interior of sealed hollow sections.

16.7.5 Chequered plates shall be painted but the details of painting shall be specified by the purchaser.

16.7.6 In case of surfaces to be welded, the steel shall not painted or metal coated within a suitable distance of any edge to be welded, if the paint specified or the metal coating is likely to be harmful to welders or impair the quality of the welds.

16.7.7 Welds and adjacent parent metal shall not be painted prior to de-slagging, inspection and approval.

16.7.8 Parts to be encased in concrete shall not be painted or oiled.

16.7.9 Contact surface in friction type connection shall not be painted in advance.

16.8 Marking

Each piece of steel work shall be distinctly marked before dispatch, in accordance with a marking diagram and shall bear such other marks as well facilitate erection.

16.9 Shop Erection

16.9.1 The steel work shall be temporarily shop erected complete or as arranged with the inspection agency so that the accuracy of fit may be checked before dispatch. The part shall be shop assembled with sufficient numbers of parallel drifts to bring and keep the parts in place.

16.9.2 In the case of parts drilled or punched, through steel jigs bushes resulting in all similar parts being interchangeable, the steelwork may be shop erected in such position as arranged with the inspection agency.

16.9.3 In case of shop fabrication using numerically controlled machine data generated by computer software (like CAD), the shop erection may be dispensed with at the discretion of the inspector.

16.10 Packing

All projecting plates or bars and all ends of members at joints shall be stiffened, all straight bars and plates shall be bundles, all screwed ends and machined surfaces shall be suitably packed and all bolts, nuts, washers and small and loose parts shall be packed separately in cases, so as to prevent damage or distortion during transit.

16.11 Inspection and Testing

16.11.1 The inspecting authority shall have free access at all reasonable items to those parts of the manufacturer's works which are concerned with the fabrication of the steelwork and shall be afforded all reasonable facilities for satisfying himself that the fabrication is being undertaken in accordance with the provisions of this standard.

16.11.2 Unless specified otherwise, inspection shall be made at the place of manufacture prior to dispatch and shall be conducted so as not to interfere unnecessarily with the operation of the work.

16.11.3 The manufacturer shall guarantee compliance with the provisions of this standard, if required to do so by the purchaser.

16.11.4 Should any structure or part of a structure be found not to comply with any of the provisions of this standard, it shall be liable to rejection. No structure or part of the structure, once rejected shall be resubmitted for test, except in cases where the purchaser or his authorized representative considers the defect as rectifiable.

16.11.5 Defects, which may appear during fabrication, shall be made good with the consent of an according to the procedure laid down by the inspecting authority.

16.11.6 All gauges and templates necessary to satisfy the inspection authority shall be supplied by the manufacturer. The inspecting authority may, at his discretion, check the test results obtained at the manufacturer's works by independent testing at outside laboratory, and should the material so tested be found to be unsatisfactory, the cost of such tests shall be borne by the manufacturer, and if found satisfactory the cost shall borne by the purchaser.

16.12 Site Erection

16.12.1 Plant and Equipment

The suitability and capacity of all plant and equipment used for erection shall be to the satisfaction of the engineer.

16.12.2 Storing and Handling

All structural steel should be so stored and handled at the site that the members are not subjected to excessive stresses and damage by corrosion due to exposure to environment.

16.12.3 Setting out

The positioning and levelling of all steelwork, the plumbing of stanchions and the placing of every part of the structure with accuracy shall be in accordance with the approved drawings and to the satisfaction of the engineer in accordance with the deviation permitted below

16.12.3.1.1 Erection tolerances

16.12.3.2 Unloaded steel structures, as erected, shall satisfy the criteria specified in Table 36 within the specified tolerance limits.

Each criterion given in the table shall be considered as a separate requirement, to be satisfied independent of any other tolerance criteria. The erection tolerances specified in Table 36 apply to the following reference points:

a) For a column, the actual center point of the column at each floor level and at the base, excluding any base-plate or cap-plate. The level of the base plate on pedestal shall be so as to avoid contact with soil and corrosive environment; and
b) For a beam, the actual center point of the top surface at each end of the beam, excluding any end-plate.

Table 36: Normal Tolerances After Erection

SN	Criterion	Permitted Deviation
i)	Deviation of distance between adjacent columns	5 mm
ii)	Inclination of a column in a multi-storey building between adjacent floor levels	$0.002\ h_s$, where h_s is the storey height
iii)	Deviation of location of a column in a multi-storey building at any floor level from a vertical line through the intended location of the column base	$0.0035 \sum h_b / n^{0.5}$, where $\sum h_b$ is the total height from the base to the floor level concerned and n is the number of storeys from the base to the floor level concerned.
iv)	Inclination of a column in a single storey building,(not supporting a crane gantry) other than a portal frame	$0.0035\ h_c$, where, h_c is the height of the column.
v)	Inclination of the column of a portal frame (not supporting a crane gantry)	Mean: $0.002\ h_c$; Individual: $0.010\ h_c$, where h_c is the height of the column

The straightness tolerances specified in Table 37 have been assumed in the derivation of the design stress for the relevant type of member. Where the curvature exceeds these values, the effect of additional curvature on the design calculations shall be reviewed.

A tension member shall not deviate from its correct position relative to the members to which it is connected by more than 3 mm along any setting axis.

Table 37: Straightness Tolerances Incorporated in Design Rules

SN	Criterion	Permitted Deviation
i)	Straightness of a column (or other compression member) between points which will be laterally restrained on completion of erection	$0.001L$ generally, and $0.002L$ for members with hollow cross-sections; where, L is the length between points which will be laterally restrained
ii)	Straightness of a compression flange of a beam, relative to the weak axis, between points, which will be laterally restrained on completion of erection.	$0.001L$ generally, and $0.002L$ for members with hollow cross-sections; where, L is the length between points which will be laterally restrained

16.12.4 Safety During fabrication and erection

16.12.4.1 All steel materials including fabricated structures, either at fabrication shop or at erection site, shall be handled only by a worker skilled in such jobs; where necessary with load tested lifting devices, having tested wire rope slings of correct size. The devises should be well maintained and operated by experienced operators.

16.12.4.2 Oxygen and acetylene cylinders and their hoses shall have distinctive colors. Cylinders should be stored in upright position in well-ventilated rooms or in open air, not exposed to flames, naked lights or extreme heats and should also be in upright position when they are being used. All gas cutting works shall be done only by experienced skilled gas cutters, equipped with gloves, boots, aprons, goggles, and good cutting sets of approved make.

16.12.4.3 While doing any welding work, it should be ensured that the welding machine is earthed and the welding cables are free from damage. The welder and his assistant shall use a face shield or head shield with a welding lens and clear cover glass and their hands, legs and bodies shall be well protected by leather gloves, shoes and aprons. Combustible materials should be kept away from the sparks and globules of molten metals generated in any arc welding. In case of welding in a confined place, it should be provided with an exhaust system to take care of he harmful gases, fumes and dusts generated.

16.12.4.4 In addition to precautions against all the hazards mentioned above, erection workers shall also be protected in the following manner:

a) All workers shall wear helmets and shall also be provided with gloves and shoes. In addition those working at heights shall use safety belts.
b) All structures shall be so braced/guyed during erection that there is no possibility of collapse before erection work is completed.
c) Warning signs such “Danger”, “Caution”, “440 Volt”, “Do not smoke”, “Look ahead”, etc. should be displayed at appropriate places.

16.12.4.5 For detailed safety precautions during erection, reference shall be made to IS 7205.

16.12.5 Field Connections

16.12.5.1.1 Field bolting

Field bolting shall be carried out with the same care as required for shop bolting.

16.12.5.2 Fillet welding

Field assembly and welding shall be executed in accordance with the requirements for shop fabrications excepting such as manifestly apply to shop conditions only. Where the steel has been delivered painted, the paint shall be removed for a distance of at least 50 mm on either side of the joint.

16.13 Painting after Erection

16.13.1 Before painting of such steel which is delivered unpainted is commenced, all surfaces to be painted shall be dry and thoroughly cleaned from all loose scale and rust, as required by the surface protection specification.

16.13.2 The specified protective treatment shall be completed after erection. All bolts heads and the site welds after de-slagging shall be cleaned. Damaged or deteriorated paint surfaces shall first be made good with the same type of paint as the shop coat. Where specified, surfaces will be in contact after site assembly, shall receive a coat of paint (in addition to any shop priming) and shall be brought together while the paint is still wet. No painting shall be sued on contact surfaces in the friction connection unless specified otherwise by the design document.

16.13.3 Where the steel has received a metal coating in the shop, this coating shall be completed on site so as to be continuous over any welds and bolts, subject to the approval of the Engineer, Painting on site may complete protection. Bolts, which have been galvanized or similarly treated, are exempted from this requirement.

16.13.4 Surface, which will be in accessible after site assembly, shall receive the full-specified protective treatment before assembly.

16.13.5 Site painting should not be done in frosty or foggy weather, or when humidity is such as to cause condensation on the surfaces to be painted.

16.14 Bedding Requirement

16.14.1 Bedding shall be carried out with Portland cement grout or mortar as described under 17.15.4 or fine cement concrete.

16.14.2 For multistoried buildings, this operation shall not be carried out until a sufficient number of bottom lengths of stanchions have been properly lined, leveled and plumbed and sufficient floor beams are in position.

16.14.3 Whatever method is employed, the operation shall not be carried out until the steelwork has been finally levelled and plumbed, stanchion bases being supported meanwhile by steel wedges or nuts; and immediately before grouting, the space under the steel shall be thoroughly cleaned.

16.14.4 Bedding of structure shall be carried out with grout or mortar, which shall be of adequate strength and shall completely fill the space to grouted and shall either be placed under pressure or by ramming against fixed supports. The grouts or mortar used shall be non-shrinking variety.

17 ANNEX A: Analysis and Design Methods

17.1 Advanced Structural Analysis and Design

Analysis

For a frame, comprising members of compact section with full lateral restraint, an advanced structural analysis may be carried out, provided the analysis can be shown to accurately model the actual behaviour of that class of frames. The analysis shall take into account the relevant material properties, residual stresses, geometrical imperfections, reduction in stiffness due to axial compression, second-order effects, section strength and ductility, erection procedures and interaction with the foundations.

Advanced structural analysis for earthquake loads shall take into account provisions of NBC 105.

Design

For the strength limit state, it shall be sufficient to satisfy the section capacity requirements of Section 8 for the members subjected to bending, of Section 7 for axial members, of Section 9 for combined forces and of Section 10 for connections. Effect of moment magnification given in Section 9, instability given in Section 7 and lateral buckling given in Section 8 need not be considered while designing the member, since advanced analysis methods directly consider these.

An advanced structural analysis for earthquake loads shall recognize that the design basis earthquake loads calculated in accordance with IS 1893 is assumed to correspond to the load at which the first significant plastic hinge forms in the structure.

Second Order Elastic Analysis and Design

17.1.1 Analysis

In a second-order elastic analysis, the members shall be assumed to remain elastic, and changes in frame geometry under the design load and changes in the effective stiffness of the members due to axial forces shall be accounted for. In a frame where the elastic buckling load factor (λcr) of the frame as determined in accordance with 10.6 is greater than 5, the changes in the effective stiffness of the members due to axial forces may be neglected.

17.1.2 Design Bending Moment

The design bending moment under factored load shall be taken as the maximum bending moment in the length of the member. It shall be determined either:

a) directly from the second-order analysis; or
b) approximately, if the member is divided into a sufficient number of elements, as the greatest of the element end bending moments; or
c) by amplifying the calculated design bending moment, taken as the maximum bending moment along the length of a member as obtained by superposition of the simple beam bending moments determined by the analysis.

For a member with zero axial force or a member subject to axial tension, the factored design bending moment shall be calculated as the moment obtained from second order analysis without any amplification.

For a member with a design axial compressive force as determined from the analysis, the factored design bending moment shall be calculated as follows:

$$M = \delta_b M_m$$

Where, δ_b = moment amplification factor for a braced member determined in accordance with 16.

17.2 Frame Instability Analysis

17.2.1 Analysis

Frame instability, as treated here, is related to the design of multi-storey rigid-jointed frames subject to side sway. The elastic critical load factor, λ_{cr} may be determined using the deflection method as given in B-3.2 or any other recognized method. This is used to calculate the amplified sway moments for elastic designs and to check frame stability in plastic designs. The elastic critical load factor, λ_{cr}, of a frame is the ratio by which each of the factored loads would have to be increased to cause elastic instability.

17.2.2 Deflection Method

An accurate method of analysis (ordinary linear elastic analysis) should be used to determine the horizontal deflections of the frame due to horizontal forces applied at each floor level, which is equal to the notional horizontal load in 4.3.6. Allowance should be made 120 IS 800:2007 for the degree of rigidity of the base as given in B-3.2 in this deflection calculation.

The base stiffness should be determined by reference to 4.3.4.

The elastic critical load factor, λ_{cr}, is calculated as:

$$\lambda_{cr} = \frac{1}{200\phi_{s,Max}}$$

Where,

$\phi_{s,Max}$ = largest value of the sway index where, ϕ_s given by:

$$\phi_s = \frac{\delta_{ui} - \delta_{Li}}{h_i}$$

Where, h_i = storey height;

δ_{ui} = horizontal deflection of the top of the storey due to the combined gravity and notional loads;

δ_{Li} = horizontal deflection of the bottom of the storey due to gravity and notional load.

17.2.3 Partial Sway Bracing

In any storey the stiffening effect of infill wall panels may be allowed for by introducing a diagonal strut in that storey of area A, given by: $A = \frac{k_3 \sum K_c}{h\left(\frac{h}{b}\right)}\left[1 + \left(\frac{h}{b}\right)^2\right]^{\frac{3}{2}}$

Where, h = storey height;

b = Width of the braced bay;

$\sum K_c$ = sum of the stiffness I/L ; of the columns in that storey;

$k_3 = \frac{h^2 \sum S_p}{80\, E \sum K_c} \leq 2$; and

$\sum S_p$ = Sum of spring stiffness (horizontal force per unit horizontal deflection of all the panels in that storey determined from: $S_p = \frac{0.6h/b}{\left(1+\left(\frac{h}{b}\right)^2\right)^2} t_p E_p$

Where, t_p = thickness of the wall panel, and

E_p = modulus of elasticity of the panel material.

18 ANNEX B: Working Stress Design

18.1 General

18.1.1 General design requirements of Section 3 shall apply in this section. Methods of structural analysis of Section 4 shall also be applied to this section. The elastic analysis method shall be used in the working stress design.

18.1.2 The working stress shall be calculated applying respective partial load factor for service/working load.

18.1.3 In load combinations involving wind or seismic loads, the permissible stresses in steel structural members may be increased by 33 percent. For anchor bolts and construction loads this increased by 33%. For anchor bolts and construction loads this increase shall be limited to 25 percent. Such an increase in allowable stress should not be considered if the load combination (such as acting along with dead load alone).

18.2 Tension members

18.2.1 Actual tensile stress

The actual tensile stress,f_t on the gross area of cross section, $A_g$if plate, angles and other tension members shall be less than or equal to the smaller value of permissible tensile stresses,f_{at}, as given below:

Actual tensile stress, $f_t = T_s/A_g$

The permissible stress, f_{at} is smaller of the values as obtained below:

a. as governed by yielding of gross section: $f_{at} = 0.6 f_y$

b. As governed by rupture of net section:

1. Plates under tension: $f_{at} = 0.69\, T_{dn}/A_g$
2. Angles under tension: $f_{at} = 0.69\, T_{dn}/A_g$

c. As governed by block shear: $f_{at} = 0.69\, T_{db}/A_g$

where, T_s = actual tension (service) load; A_g = gross area,

T_{dn} = design strength under working intension of respective plate/angle

T_{db} =design tension block shear strength in of respective plate/angle

18.3 Compression Members

18.3.1.1 Actual Compressive stress

The actual compressive stress, f_c at working (service) load, P_s of a compression member shall be less than the permissible compressive stress, f_{ac} as given below:

Actual Compressive stress, $f_c = P_s/A_e$

The permissible compressive stress, $f_{ac} = 0.60\, f_{cd}$

where, A_e = effective sectional area defined in 7.3.2 ; f_{cd} = design compressive stress defined in 7.1.2.1**.**

18.3.2 Design Details:

Design of compressive members shall conform to **7.3.**

18.3.3 Column Bases

The provisions of 7.5 shall be followed for the design of column bases, except that the thickness of a simple column base, t_s shall be calculated as :$t_s = \sqrt{3w(a^2 - 0.3b^2)/f_{bs}}$

where, w =uniform pressure from below on the slab base due to axial pressure

a, b =larger and smaller projection of the slab base beyond the rectanglecircumscribing the column

f_{bs} =permissible bending stress in column base equal to 0.75 f_y

18.3.4 Angle Struts

Provisions of 7.4 shall be used for design of angle struts, except that the limiting actual stresses shall be calculated in accordance with **11.3.1.**

18.3.5 Laced and Battened Columns

The laced and battened columns shall be designed in accordance with **7.6 and 7.7**, except that the actual stresses shall be less than the permissible stresses give in **11.3.1.**

18.4 Members subjected to Bending

18.4.1 Bending Stresses

The actual bending tensile and compressive stresses, f_{bt}, f_{bc} at working (service) load moment, M_S of a bending member shall be less than or equal to the permissible bending stresses, f_{abt}, f_{abc} respectively, as given herein. The actual bending stresses shall be calculated as:$f_{bc} = M_s/Z_{ec}$ and $f_{bt} = M_s/Z_{et}$

The permissible bending stresses, f_{abt}, f_{abc} shall be the smaller of the values obtained from the following:

a. Laterally supported beams and beams bending about the minor axis:
   1. Plastic and compact sections: $f_{abc} or\ f_{abt} = 0.66 f_y$
   2. Semi-compact sections: $f_{abc} or\ f_{abt} = 0.60\ f_y$

b. Laterally unsupported beams subjected to major axis bending:

$$f_{abc} = 0.60 M_d/Z_{cc}$$
$$f_{abt} = 0.60 M_d/Z_{ct}$$

c. Plates and solid rectangles bending about minor axis:

$$f_{abc} = f_{abt} = 0.75 f_y$$

where, Z_{ec}, Z_{et} = elastic section modulus for the cross section with respect to the extreme Compression and tensions fibers, respectively;

f_y =yield stress of the section; and

M_d =design bending strength of a laterally unsupported beam bent about major axis, calculated in accordance with **8.2.2.**

18.4.2 Shear Stress in Bending Members

The actual shear stress, τ_b at working load, V_s of a bending member shall be less than or equal to the permissible shear stress, τ_{ab} given below:

Actual shear stress, $\tau_b = V_s/A_v$

The permissible shear stress is given by:

a. When subjected to pure shear:$\tau_{ab} = 0.40 f_y$

b. When subjected to shear buckling: $\tau_{ab} = 0.70 \frac{V_n}{A_v}$

where, V_n =design shear strength as given in **8.4.2.2(a)**,and
A_v =shear area of the cross-section as given in **8.4.1**

18.4.3 Plate girder

Provisions of **8.3, 8.4, 8.5,8.6 and 8.7** shall apply, for the design of plate girder, except that the allowable stresses shall conform to **11.4.1 and 11.4.2.**

18.4.4 Box Girder

In design of box girder the provision of 8.8 shall apply, except that the allowable bending stresses shall conform to **11.4.1**

18.5 Combined Stresses

18.5.1 Combined Bending and Shear

Reduction in allowable moment need not be considered under combined bending and shear.

18.5.2 Combined bending and axial compression force

Members subjected to combined axial compression and bending shall be so proportioned to satisfy the following requirements:

a. Member stability requirement

$$\frac{f_c}{f_{acy}} + \frac{0.6K_y C_{my} f_{bcy}}{f_{abcy}} + \frac{K_{LT} f_{bcx}}{f_{abcx}} \leq 1.0$$

$$\frac{f_c}{f_{acx}} + \frac{0.6K_y C_{my} f_{bcy}}{f_{abcy}} + \frac{K_x C_{mz} f_{bcx}}{f_{abcx}} \leq 1.0$$

Where,

C_{my}, C_{mx} =equivalent uniform moment factor as per **table 18**

f_c =applied axial compressive stress under service load

f_{bcy}, f_{bcx} =applied compressive stresses due to bending about the major (y) and minor axis (x) of member respectively

f_{acy}, f_{acx} =allowable axial compressive stresses due to buckling about the minor and major axis of member respectively

f_{abcy}, f_{abcx} =applied bending compressive stresses due to bending about the minor and major axis of member respectively

$$K_y = 1 + (\lambda_y - 0.2)n_y \leq 1 + 0.8n_y; \quad K_x = 1 + (\lambda_x - 0.2)n_x \leq 1 + 0.8n_x,$$

$$1 - \frac{0.1\lambda_{Lt} n_y}{(C_{mLT} - 0.25)} \geq 1$$

$$K_{LT} = -\frac{0.1n_y}{(C_{mLT} - 0.25)}$$

where,

n_y, n_x = Ratio of actual applied axial stress to the allowable axial stress for buckling about the y and x axis, respectively;

C_{mLT}= equivalent uniform factor; λ_{LT}= non-dimensional ratio **(see 8.2.2).**

b. Member strength requirement

At a support the values f_{abcy} *and* f_{abcz} shall be calculated using laterally supported member and shall satisfy;

$$\frac{f_c}{0.6f_y} + \frac{f_{bcy}}{f_{abcy}} + \frac{f_{bcx}}{f_{abcx}} \leq 1.0$$

18.5.3 Combined Bending and Axial Tension

Members subjected to both axial tension and bending shall be proportioned so that the following condition is satisfied: $\frac{f_t}{f_{at}} + \frac{f_{bty}}{f_{abty}} + \frac{f_{btx}}{f_{abtx}} \leq 1.0$

where, f_{abty}, f_{abtz} =permissible tensile stresses under bending about minor and major axis when bending alone is acting as given in 11.4.1

18.5.4 Combined Bearing, Bending and Shear Stress

Where a bearing stress is combined with tensile or compressive stress, bending and shear stresses under the most unfavorable conditions of loading, the equivalent stress, f_e obtained from the following formula, shall not exceed $0.9 f_y$.

$$f_e = \sqrt{f_b^2 + f_p^2 + f_b^2 f_p^2 + 3\tau_b^2}$$

where, τ =actual shear stress; f_t =actual tensile stress; f_y =yield stress,and f_p =actual bearing stress

The value of permissible bending stresses f_{bcy} and f_{bcz} to be used in the above formula shall each be lesser of the values of the maximum allowable stresses f_{abc} and f_{abt} in bending about appropriate axis.

18.6 Connections

18.6.1 All design provisions of **<u>section 10</u>**, except for the actual and permissible stress calculations, shall apply.

18.6.2 Actual stresses in fasteners

18.6.2.1 Actual stress in bolt in shear, f_{sb} should be less than permissible stress of the bolt, f_{asb} as given below:

The actual stress in bolt in shear, $f_{sb} = V_{sb}/A_{sb}$

The permissible stress in bolt in shear, $f_{asb} = 0.6\, V_{nsb}/A_{sb}$

where, V_{sb} = actual shear force under working (service) load,

V_{nsb} = nominal shear capacity of the bolt as given in **10.3.3;**

A_{sb} = nominal plain shank area of the bolt

18.6.2.2 Actual stress of the bolt in bearing on any plate, f_{pb} should be less than or equal to the permissible bearing stress of the bolt/plate, f_{apb} as given below:

Actual stress of bolt in bearing on any plate: $f_{pb} = \frac{V_{sb}}{A_{pb}}$

The permissible bearing stress of the bolt/plate: $f_{apb} = \frac{0.60 \quad npb}{A_{pb}}$

where, V_{npb} =nominal bearing capacity of a bolt on any plate as in **<u>10.3.4</u>**

A_{pb} =nominal bearing area of the bolt on any plate

18.6.2.3 Actual tensile stress of the bolt, f_{tb}, should be less than or equal to permissible tensile stress of the bolt, f_{atb} as given below:

Actual tensile stress of the bolt, $f_{tb} = T_s/A_{sb}$

The permissible tensile stress of the bolt: $f_{atb} = 0.60 T_{ab}/A_{sb}$

where, T_s =tension in bolt under working (service)load

T_{nb} =design tensile capacity of bolt as given in **10.3.5**

A_{sb} =nominal plain shank area of the bolt

18.6.2.4 Actual compressive or tensile or shear stress of a weld, f_w should be less than or equal to permissible stress of the weld, f_{aw} as given below:

The permissible stress of the weld, $f_{aw} = 0.6 f_{wn}$

where, f_{wn} =nominal shear capacity of the weld as calculated in **10.5.7.1.1**

18.6.2.5 If the bolt is subjected to combined shear and tension, the actual shear and axial stresses calculated in accordance with **11.6.2.1** and **11.6.2.3** do not exceed the respective permissible stresses f_{asb} and f_{atb} then the expression given below should satisfy;

$$\left[\frac{f_{sb}}{f_{asb}}\right]^2 + \left[\frac{f_{tb}}{f_{atb}}\right]^2 \leq 1.0$$

where, f_{sb}, f_{tb} =actual shear and tensile stresses respectively,and

f_{asb}, f_{atb} =permissible shear and tensile stresses respectively

18.6.3 Stresses in Welds

18.6.3.1 Actual stresses in the throat are of fillet welds shall be less than or equal to permissible stresses, f_{aw} as given as: $f_{aw} = 0.4 f_y$

18.6.3.2 Actual stresses in the butt welds shall be less than the permissible stress as governed by the parent metal welded together.

19 ANNEX C: Evaluation of Existing Structures

This appendix applies to the evaluation of the strength and stiffness of existing structures by structural analysis. Load testing in accordance with this appendix applies to static vertical gravity load effects.

19.1 General Provisions

These provisions shall be applicable where 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.

19.2 Material Properties

For evaluations in accordance with this appendix, steel grades other than those listed in section 2 are permitted.

19.2.1 Determination of Required Tests

The engineer in charge shall determine the specific tests needed to ascertain the tensile properties, chemical composition, weld metal quality, and tensile strength of the bolt as described in more detail under the following sub-headings and specify the locations where they are required. The use of applicable project records is permitted to reduce or eliminate the need for testing.

19.2.2 Tensile Properties

The tensile properties of members shall be established for use in evaluation by structural analysis (Section 5.3). Such properties shall include the yield stress, tensile strength, and percentage of elongation. Certified material test reports or certified reports of tests made by the fabricator are permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with IS 1608, IS 1786, and IS 4293 from samples taken from components of the structure

19.2.3 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. Results from certified material test reports or certified reports of tests made by the fabricator are permitted for this purpose.

19.2.4 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 final weld quality are not met, the engineer in charge shall determine if remedial actions are required.

19.2.5 Bolts

Representative samples of bolts shall be visually inspected to determine markings and classifications. Where it is not possible to classify bolts by visual inspection, representative samples shall be taken and tested to determine tensile strength in accordance with IS 1608, and the bolt classified accordingly.

19.3 Evaluation by Structural Analysis

19.3.1 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, it is permitted to determine such dimensions from applicable project design or fabrication documents with field verification of critical values.

19.3.2 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 **3.3 Loads and Loads Combinations**.

19.3.3 Serviceability Evaluation

Where required, the deformations at service loads shall be calculated and reported.

5.5. Evaluation Report

After the evaluation of an existing structure has been completed, the engineer in charge shall prepare a report documenting the evaluation. The report shall indicate the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design documents, material test reports, and auxiliary material testing shall also be reported. The report shall indicate whether the structure, including all members and connections, can withstand the load effects.

20 ANNEX D: Design of Composite Structures

20.1 Scope

20.1.1 This document allows the design and construction of composite structural systems that combine structural steel with either cast in-situ or precast concrete, ensuring they function together as a unified element.

20.1.2 It is applicable to both simply supported and continuous composite beams and slabs, along with their associated column systems.

20.2 References

20.2.1 Steel design shall comply with the materials, workmanship, and specifications outlined in NBC 111 (this document). Concrete design shall conform to the provisions of IS 456:2000. For earthquake-resistant design, guidance from NBC 105:2020 shall be followed.

20.2.2 The use of materials not covered by this standard or IS 456:2000 may be permitted with the approval of the engineer in charge.

20.3 Design of Composite Structures

20.3.1 This document permits the design and construction of composite structures only when using the limit state design methodology.

20.3.2 Composite steel–concrete structures designed and constructed in accordance with IS 11384:2022 shall be deemed acceptable under the provisions of this document.

21 ANNEX E: Design Against Floor Vibration

21.1 General

Floors with longer spans, lighter construction, and lower inherent damping are susceptible to vibrations under normal human activity. The natural frequency of the floor system corresponding to its lowest vibration mode, along with its damping characteristics, are key parameters in floor vibration performance. Open web steel joists (trusses) or steel beams supporting concrete decks may experience vibration issues due to walking. Fatigue, floor system overloading, and vibrations from rhythmic activities such as aerobics or dance classes are beyond the scope of this Annex.

21.2 Annoyance Criteria

In the frequency range of 2 to 8 Hz—where people are most sensitive to vibration—the threshold level is approximately 0.5 percent of g, with g being the acceleration due to gravity. Continuous vibrations are generally more disturbing than those that decay over time due to damping. Floor systems with a natural frequency below 8 Hz for areas supporting rhythmic activities, and below 5 Hz for areas subject to normal human activity, should be avoided.

21.3 Floor Frequency

The fundamental natural frequency can be estimated by assuming full composite action, even in non-composite construction. This frequency, f_1, for a simply supported one way system is given by:

$$f_1 = 156\sqrt{EI_T/WL^4}$$

where, E = modulus of elasticity of steel, MPa

I_T = transformed moment of inertia of the one way system (in terms of equivalent steel) assuming the concrete flange of width equal to the spacing of the beam to be effective, in mm⁴

L = span length, in mm

W = dead load of the one way joist, in N/mm

If the one way joist system is supported by a flexible beam running perpendicular with the natural frequency f_2, the floor frequency may be reduced to f_r, given by:

$$\frac{1}{f_r^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2}$$

21.4 Damping

The percentage of critical damping may be assumed approximately as given below:

SN	System	Critical Damping Percent
i	Fully composite construction	2
ii	Bare steel beam and concrete deck	3–4
iii	Floor with finishes, false ceiling, fire proofing, ducts furniture	6
iv	Partitions not located along a support or not spaced farther apart than 6 m and partitions oriented in orthogonal directions	Up to 12

21.5 Acceleration

The peak acceleration a_p from heel impact for floors of spans greater than 7 m and natural frequency f_1 less than 10 Hz may be calculated as:

$$\frac{a_o}{g} = \frac{600 f_r}{W}$$

where, W = total weight of floors plus contents over the span length, expressed in equivalent floor width (b), in N

b = 40 t_s(20 t_s if one way hang is only on one side of the beam)

t_s = equivalent thickness of the slab, averaging thickness of steel slab and ribs; and

g = acceleration due to gravity

22 ANNEX F: Determination of Effective Lengths of Columns

22.1 Method for Determining Effective Length of Columns in Frames

In the absence of a more exact analysis, the effective length of columns in framed structures may be obtained by multiplying the actual length of the column between the centres of laterally supporting members (beams) given in Fig. 27 and Fig. 28 with the effective length factor K, calculated by using the equations given below, provided the connection between beam and column is rigid type:

a) Non-sway frames (Braced frame) [(see 11.1.2 (a)]

A frame is designated as non-sway frame if the relative displacement between the two adjacent floors is restrained by bracings or shear walls (see 11.1.2). The effective length factor, K, of column in non-sway frames is given by (see Fig. 27):

$$K = \frac{[1 + 0.145(\beta_1 + \beta_2) - 0.265\beta_1\beta_2]}{[2 - 0.364(\beta_1 + \beta_2) - 0.247\beta_1\beta_2]}$$

b) Sway frames (Moment Resisting Frames) [see 11.1.2 (b)]

The effective length factor K, of column in sway frames is given by (see Fig. 28):

$$K = \left[\frac{1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2}{1 - 0.8(\beta_1 + \beta_2) + 0.6\beta_1\beta_2}\right]^{0.5}$$

Where,

β_1, β_2 are given, $\beta = \frac{\sum K}{\sum K_c + \sum K_b}$

$K_c, K_b =$ Effective flexural stiffness of the columns and beams meeting at the joint at the ends of the columns and rigidly connected at the joints, and these are calculated by: $K = C(I/L)$

$I =$ moment of inertia of the member about an axis perpendicular to the plan of the frame,

L = length of the member equal to centre to centre distance of the intersecting member, and

C = correction factor as shown in Table 35.

Table 38: Correction Factors for Effective Flexural Stiffness

SN	Far End Condition	Correction Factor, C	
		Braced Frame	**Unbraced Frame**
i)	Pinned	$1.5(1 - n^*)$	$1.5(1 - n^*)$
ii)	Rigidly connected to column	$1.0(1 - n^*)$	$1.0(1 - 0.2n^*)$
iii)	Fixed	$2.0(1 - 0.4n^*)$	$0.67(1 - 0.4n^*)$
Note: $n^* = \frac{P}{P_e}$ where, $P_e =$ elastic buckling load, and $P =$ applied load			

Special literature shall be followed for stepped columns.

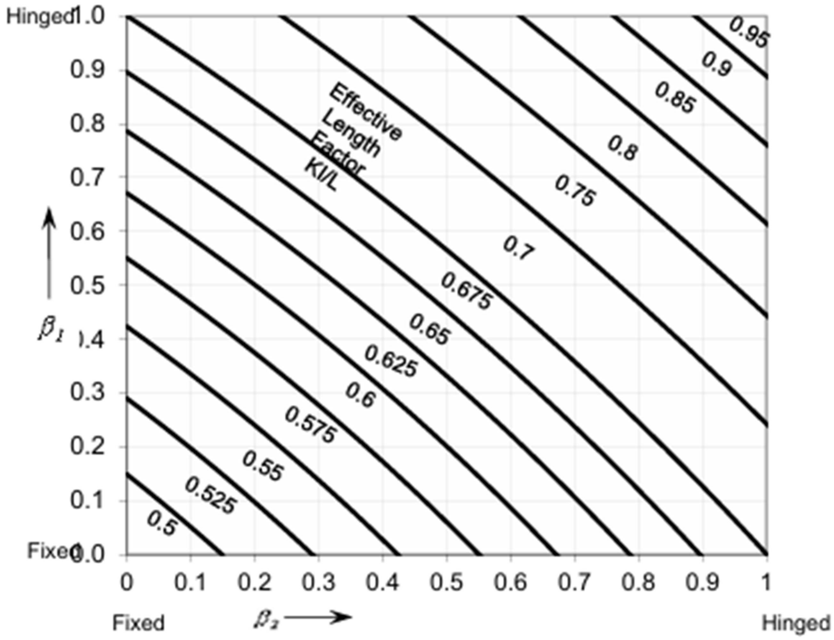


FIG. 27 COLUMN EFFECTIVE LENGTH FACTOR - NON SWAY FRAME

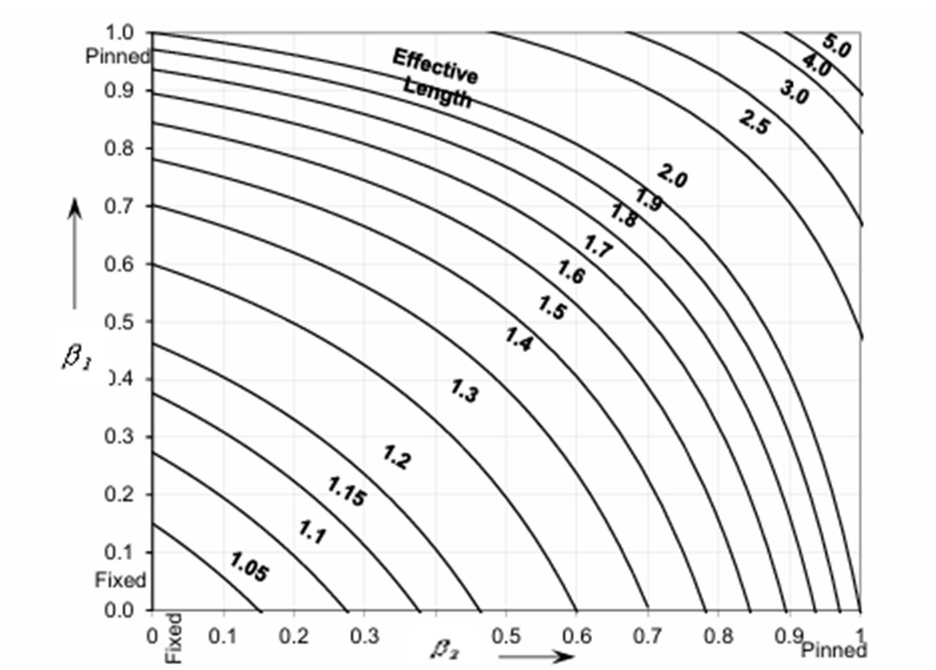


FIG. 28 COLUMN EFFECTIVE LENGTH FACTOR - SWAY FRAME

(*Clause* B-2.1)

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23 ANNEX G: Elastic Lateral Torsional Buckling

23.1 Elastic critical moment

23.1.1 General

The elastic critical moment is affected by the following:

a) Moment gradient in the unsupported length,
b) Boundary conditions at the lateral support points,
c) Non-symmetric and non-prismatic nature of the member, and
d) Location of transverse load with respect to shear centre.

The boundary conditions at the lateral supports have two components:

1) Torsional restraint – where the cross section is prevented from rotation about the shear centre, and

2) Warping restraint – where the flanges are prevented from rotating in their own plane about an axis perpendicular to the flange.

The elastic critical moment corresponding to lateral torsional buckling of a doubly symmetric prismatic beam subjected to uniform moment in the unsupported length and torsionally restraining lateral supports is given by:

$$M_{cr} = \frac{\pi^2 E I_y}{(L_{LT})^2}\left[\frac{I_w}{I_y} + \frac{G I_t (L_{LT})^2}{\pi^2 E I_y}\right]^{0.5}$$

Where, I_y, I_w, I_t = moment of inertia about the minor axis, warping constant and St.Venant's torsion constant of the cross section, respectively (see properties of sections);

G = modulus of rigidity; and

L_{LT} = effective length against lateral torsional buckling.

This equation in simplified form for I-section has been presented in **15.2.2.1.**

While the simplified equation is generally on the safe side, there are many situations where this may be very conservative. More accurate calculation of the elastic critical moment for general case of unsymmetrical sections, tapered members, loading away from shear centre and beams with moment gradient can be obtained from specialist literature, by using an appropriate computer program or equations given below.

23.2 Elastic Critical Moment of a Section Symmetrical About Minor Axis

In case of a beam which is symmetrical only about the minor axis, and bending about major axis, the elastic critical moment for lateral torsional buckling is given by the general equation below,

$$M_{cr} = c_1 \frac{\pi^2 E I_y}{(L_{LT})^2}\left\{\left[\left(\frac{K}{K_w}\right)^2 \cdot \frac{I_w}{I_y} + \frac{G I_t (L_{LT})^2}{\pi^2 E I_y} + \left(c_2 y_g - c_3 y_j\right)^2\right]^{0.5} - \left(c_2 y_g - c_3 y_j\right)\right\}$$

Where c_1, c_2, c_3 = factors depending upon the loading and end restraint conditions **(see Table 42)**;

K = effective length factors of the unsupported length accounting for boundary conditions at the end lateral supports. The effective length factor K varies from 0.5 for complete restraint against rotation about weak axis to 1.0 for free rotate about weak axis, with 0.7 for the case of one end fixed and other end free. It is analogous to the effective length factors for compression members with end rotational restraint;

K_w = warping restraint factor. Unless special provisions to restrain warping of the section at the end lateral supports are made, K_w should be taken as 1.0;

$y_g = y$ distance between the point of application of the load and the shear centre of the cross section and is positive when the load is acting towards the shear centre from the point of application;

$y_j = y_s - 0.5 \int_A \frac{(z^2+y^2)y dA}{I_z}$;

$y_s =$ coordinate of the shear centre with respect to centroid, positive when the shear centre is on the compression side of the centroid; and

$y, z =$ coordinates of the elemental area with respect to centroid of the section.

yj can be calculated by using the following approximation:

a) Plain flanges:

$y_j = 0.8 \frac{(2\beta_f - 1)h_y}{2.0}$ (when $\beta_f > 0.5$)

$y_j = 1.0 \frac{(2\beta_f - 1)h_y}{2.0}$ (when $\beta_f \leq 0.5$)

b) Lipped flanges:

$y_j = 0.8(2\beta_f - 1)\left(1 + \frac{h_L}{h}\right).(h_y/2)$ (when $\beta_f > 0.5$)

$y_j = (2\beta_f - 1)\left(1 + \frac{h_L}{h}\right).(h_y/2)$ (when $\beta_f \leq 0.5$)

where
$h_L =$ height of the lip,
$h =$ overall height of the section,
$h_y =$ distance between shear centre of the two flanges of the cross section, and
$\beta_f = I_{fc}/(I_{fc} + I_{ft})$ where I_{fc}, I_{ft} are the moment of inertia of the compression and tension flanges, respectively, about the minor axis of the entire section.
$I_t =$ St. Venant's Torsion constant, given by:

$I_t = \sum b_i t_i^3/3$ (for open section)

$I_t = 4A_e^2/\sum(b/t)$ (for hollow section)

where
$A_e =$ area enclosed by the section, and
$b, t =$ breadth and thickness of the elements of the section, respectively.
$I_w =$ The warping constant, given by:

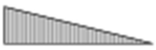
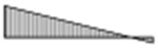
$= (1 - \beta_f)\beta_f I_y h_y^2$ (for I-sections mono-symmetric about weak axis)

= 0 (for angle, Tee, narrow rectangle section and approximately for hollow sections)

Table 42 Constants c_1, c_2, and c_3

(*Clause* D-1.2)

Loading and Support Conditions	Bending Moment Diagram	Value of K	Constants		
			c_1	c_2	c_3
(1)	(2)	(3)	(4)	(5)	(6)
	ψ = +1 	1.0	1.000		1.000
		0.7	1.000	---	1.113
		0.5	1.000		1.144

	ψ= + ¾ 	1.0	1.141		0.998
		0.7	1.270	---	1.565
		0.5	1.305		2.283
	ψ = + ½ 	1.0	1.323		0.992
		0.7	1.473	---	1.556
		0.5	1.514		2.271
	ψ = + ¼ 	1.0	1.563		0.977
		0.7	1.739	---	1.531
		0.5	1.788		2.235
	ψ = 0 	1.0	1.879		0.939
		0.7	2.092	---	1.473
		0.5	2.150		2.150
	ψ = - ¼ 	1.0	2.281		0.855
		0.7	2.538	---	1.340
		0.5	2.609		1.957
	ψ = - ½ 	1.0	2.704		0.676
		0.7	3.009	---	1.059
		0.5	3.093		1.546
	ψ = - ¾ 	1.0	2.927		0.366
		0.7	3.009	---	0.575
		0.5	3.093		0.837
	ψ = - 1 	1.0	2.752		0.000
		0.7	3.063	---	0.000
		0.5	3.149		0.000

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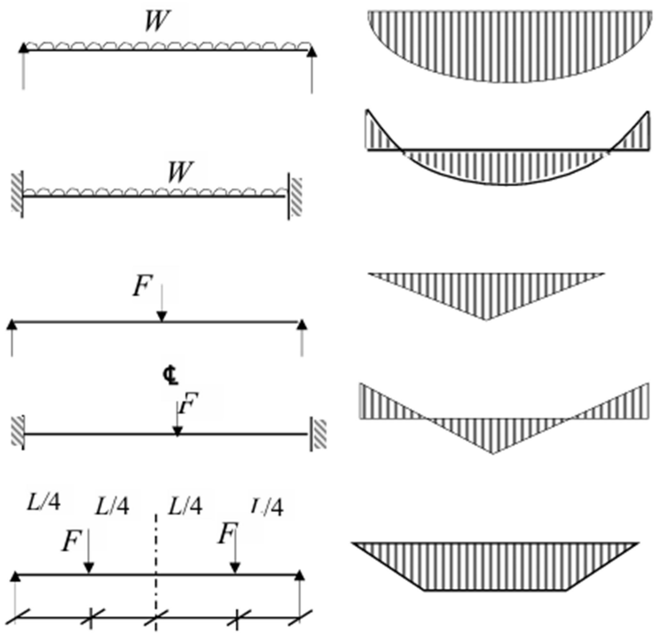

1.0	1.132	0.459	0.525
0.5	0.972	0.304	0.980
1.0	1.285	1.562	0.753
0.5	0.712	0.652	1.070
1.0	1.365	0.553	1.730
0.5	1.070	0.432	3.050
1.0	1.565	1.267	2.640
0.5	0.938	0.715	4.800
1.0	1.046	0.430	1.120
0.5	1.010	0.410	1.890

24 ANNEX H: Connections

24.1 General

The requirements for the design of splice joints and beam-to-column connections, along with the corresponding design recommendations, are outlined below.

24.2 Beam Splices

24.2.1 For rolled section beam splices located away from the point of maximum moment, it may be assumed that the flange splice carries all the moment and the web splice carries the shear (see Fig. 30). However, in the case of a deep girder, the total moment may be divided between the flange and the web in accordance with the stress distribution. The web connection should then be designed to resist its share of moment and shear. Even web splice is designed to carry only shear force, the moment about the centroid of the bolt group on either side of the splice should be designed for moment due to eccentricity.

24.2.2 Flange joints should preferably not be located at points of maximum stress. Where splice plates are used, their area shall not be less than 5 percent in excess of the area of the flange element spliced; and their centre of gravity shall coincide, as nearly as possible with that of the element spliced. There shall be enough fasteners on each side of the splice to develop the load in the element spliced plus 5 percent but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced. Wherever possible in welded construction, flange plates shall be joined by complete penetration butt welds. These butt welds shall develop the full strength of the plates. Whenever the flange width or thickness changes at the splice location, gradual transition shall be made in the width/thickness of the larger flange.

Figure 31: Beam splices

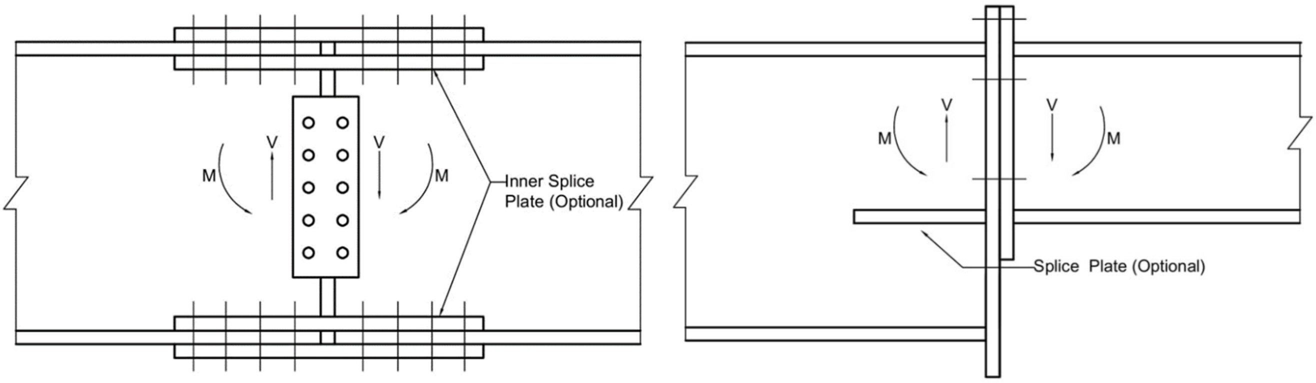

Figure 32: Beam Splices

24.2.3 When beam splice is located at the point of inflection of a continuous beam, the flange splicing requirement given above may be relaxed appropriately.

24.3 Column Splice

24.3.1 Where the ends of compression members are faced for bearing over the whole area, they shall be spliced to hold the connected parts aligned. The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing (see Fig. 31).

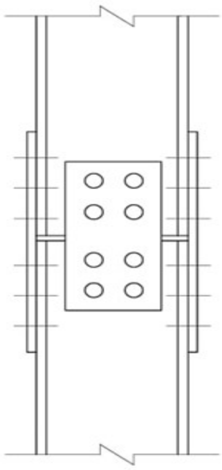

Figure 33: Column Splices

24.3.2 Where such members are not faced for complete bearing the splices shall be designed to transmit all the forces to which the members is subjected at the splice location.

24.3.3 Wherever possible, splices shall be proportioned and arranged so that centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members joined, in order to avoid eccentricity; but where eccentricity is present in the joint, the resulting stress considering eccentricity shall be provided for.

24.3.4 If a column flange is subjected to significant tension or if the faces are not prepared for bearing, or if full continuity is required without slip, only HSFG bolts shall be used.

24.4 Beam-to-Column Connections

24.4.1 Simple Connections

Simple connections are typically designed to transmit only shear and are commonly used in steel frames where the beams frame into columns or walls with greater stiffness. In such cases, a separate system must be in place to carry moment or lateral loads. The types of simple connections shown in Figure 34: Classification of Connections according to Bjorhovde are commonly used in framed construction and must be checked only for shear transfer from beam to column.

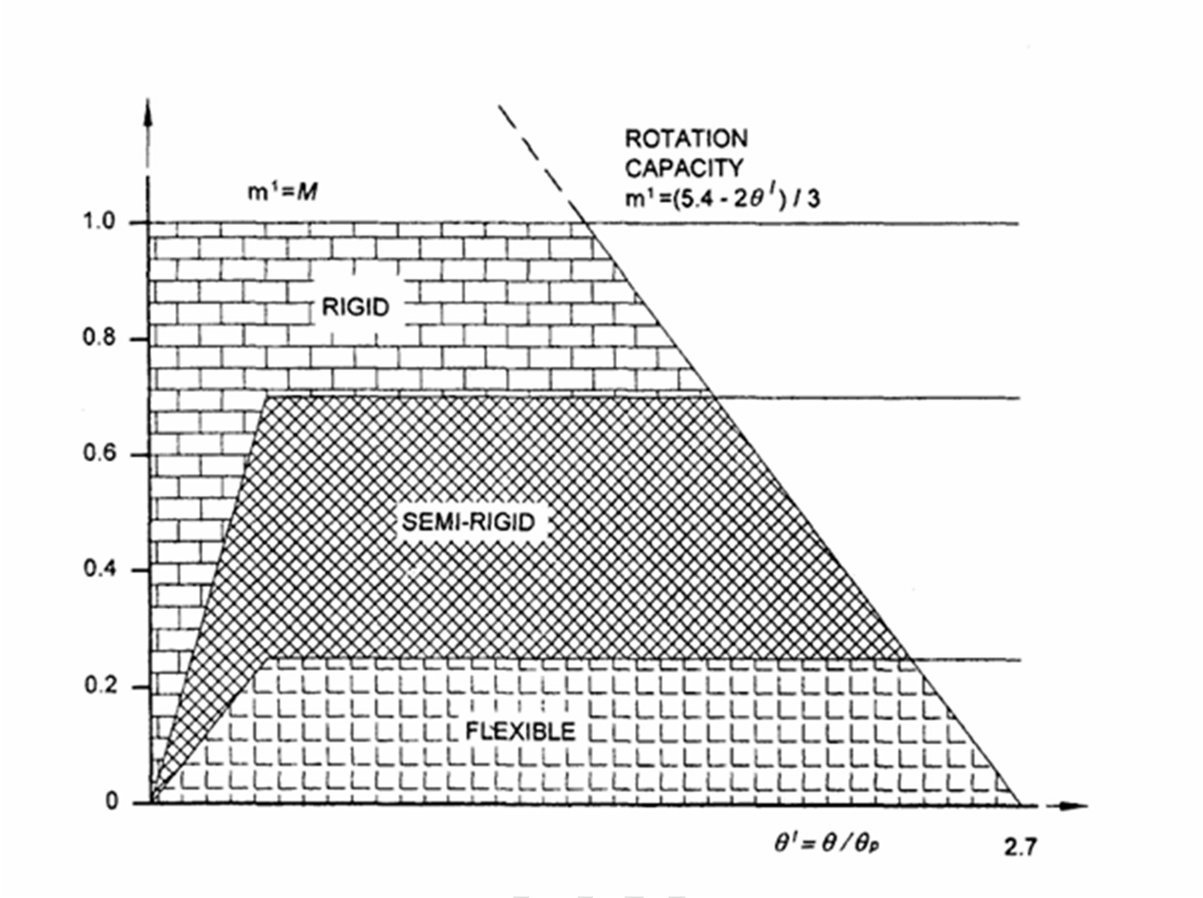

Figure 34: Classification of Connections according to Bjorhovde

24.4.2 Rigid Connections

In high-rise and slender structural systems, stiffness requirements are more stringent due to wind and seismic demands. Rigid connections are intended to transfer moments from the beam to the column while keeping joint deformations minimal. These connections are vital for structural stability and for resisting lateral forces. The examples illustrated in Figure 34: Classification of Connections according to Bjorhovde can be considered rigid when used in moment-resisting frames and must be designed to handle both shear and moment transfer. Fully welded connections are also used where complete moment continuity is required.

24.4.3 Semi-Rigid Connections

Semi-rigid connections represent a middle ground between simple and rigid connections. Although they cannot carry the same level of rotational rigidity as rigid joints, they still provide more resistance than simple shear connections. These types of joints are suitable where moderate moment transfer is needed and where some joint deformation is acceptable.

The moment-rotation behavior of semi-rigid connections must be determined through experimental data or by using relationships from specialized design literature. The simplest approach is to represent the connection as a spring element—either bilinear or non-linear—to simulate its moment-rotation characteristics. rotation characteristics. The classification proposed by Bjorhovde combined with the Frey-Morris model can be used with convenience to model semi-rigid connections, as given in the next section.

24.4.3.1 Connection Classification

Connections are categorized based on their ultimate strength or their initial rotational stiffness. This classification draws from Bjorhovde's work. It considers a non-dimensional moment parameter ($\bar{m} = M_u/M_\beta$) and a non-dimensional rotation parameter ($\bar{\theta} = \theta_u/\theta_p$), where θ_u is the plastic rotation. Bjorhovde's method references the beam span to depth ratio as a basis. The classification boundaries for various connection types are outlined in Table 39 and illustrated graphically in Figure 35.

Table 39: Connection Classification Limits

SN	Nature of the connection	In Terms of Strength	In Terms of Stiffness
i)	Rigid connection	$m^1 \geq 0.7$	$m^1 \geq 2.5\,\theta^1$
ii)	Semi-rigid connection	$0.7 > m^1 > 0.2$	$2.5\theta^1 > m^1 > 0.5\,\theta^1$
iii)	Flexible connection	$m^1 \leq 0.2$	$m^1 \leq 0.5\theta^1$

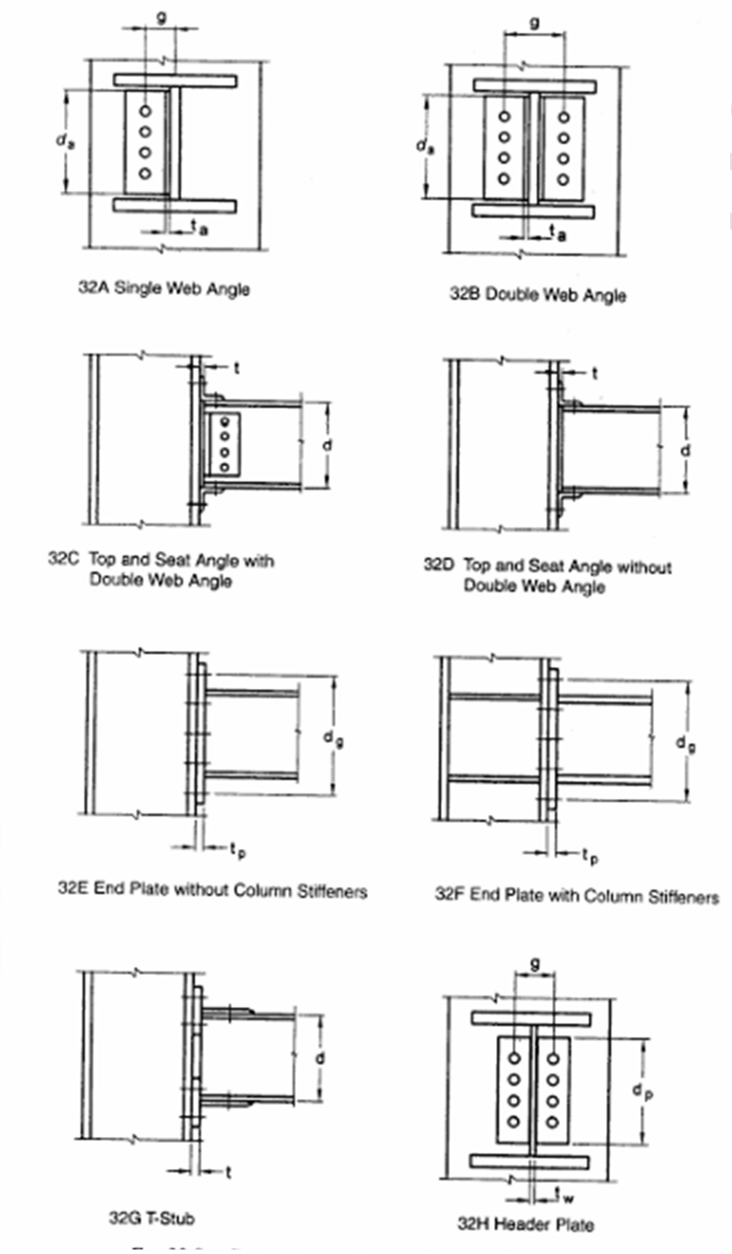

Figure 35: Size Parameter for Various Types of Connections

24.4.3.2 Connection Models

The Frye-Morris model presents a polynomial equation for the moment-curvature behavior of semi-rigid connections:

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5$$

where, M = moment at the joint, in kNm;

K = standardization factor based on connection type and geometry;

C_1, C_2, C_3 = curve-fitting constants

Table 40: Connection Constants in Frye-Morris Model

SN	Type	Connection Type	Curve-fitting Constants	Standarization
i)	A	Single web angle connection	$C_1 = 1.91 \times 10^4$ $C_2 = 1.30 \times 10^{11}$ $C_3 = 2.70 \times 10^{17}$	$K = d_a^{-2.4} t_c^{-1.81} g^{0.15}$
ii)	B	Double web angle connection	$C_1 = 1.64 \times 10^3$ $C_2 = 1.03 \times 10^{14}$ $C_3 = 8.18 \times 10^{25}$	$K = d_a^{-2.4} t_c^{-1.81} g^{0.15}$
iii)	C	Top and seat angle connection with double web angle	$C_1 = 2.24 \times 10^{-1}$ $C_2 = 1.86 \times 10^4$ $C_3 = 3.23 \times 10^8$	$K = d_a^{-1.287} t_a^{-1.128} t_c^{-0.415} l_a^{-0.694} (g - 0.5\, d_b)^{1.35}$
iv)	D	Top and seat angle connection without double web angle	$C_1 = 1.63 \times 10^3$ $C_2 = 7.25 \times 10^{14}$ $C_3 = 3.31 \times 10^{23}$	$K = d^{-1.5} t_a^{-0.5} l_a^{-0.7} d_p^{-1.1}$
v)	E	End plate connection without column stiffeners	$C_1 = 1.78 \times 10^4$ $C_2 = -9.55 \times 10^{16}$ $C_3 = 5.54 \times 10^{29}$	$K = d_g^{-2.4} t_p^{-0.4} t_f^{-1.5}$
vi)	F	End plate connection with column stiffener	$C_1 = 2.60 \times 10^2$ $C_2 = 5.37 \times 10^{11}$ $C_3 = 1.31 \times 10^{22}$	$K = d_g^{-2.4} t_p^{-0.6}$
vii)	G	T-stub connection	$C_1 = 4.05 \times 10^2$ $C_2 = 4.45 \times 10^{13}$ $C_3 = -2.03 \times 10^{23}$	$K = d^{-1.5} t_f^{-0.5} l_t^{-0.7} d_b^{-1.1}$
viii)	H	Header plate connection	$C_1 = 3.87$ $C_2 = 2.71 \times 10^5$ $C_3 = 6.06 \times 10^{11}$	$K = t_p^{-1.6} g^{1.6} d_b^{-2.3} t_w^{-0.5}$
Where, d = depth of beam, in mm. da= depth of the angle, in mm. db= diameter of the bolt, in mm. dg= center to center of the outermost bolt of the end plate connection, in mm. g = gauge distance of bolt line, in mm. ta = thickness of the top angle, in mm. tc = thickness of the web angle, in mm. tf = thickness of flange T-stub connector, in mm. tw= thickness of web of the beam in the connection, in mm. tp= thickness of end plate, header plate, in mm. la= length of the angle, in mm. lt = length of the T-stub connector, in mm. NOTE – For preliminary analysis using a bilinear moment curvature relationship, the stiffness given in Table 45 may be assumed depending on the type of connection. The values are based on the secant stiffness at a rotation of 0.01 radian and typical dimension of connecting angle and other components as given in the table.				

Table 40 provides the values of these curve-fitting constants and standardization factors for the Frye-Morris model (note: all sizes are in mm, as shown in Figure 34: Classification of Connections according to Bjorhovde).

Table 41: Secant Stiffness

SN	**Type of Connection**	**Dimension (mm)**	**Secant Stiffness (kNm/radian)**
i)	Single web connection angle	$d_a = 250, t_a = 10, g = 35$	1150

ii)	Double web-angle connection	$d_a = 250, t_a = 10, g = 77.5$	4450
iii)	Top and seat angle connection without double web angle connection	$d_a = 300, t_a = 10, l_a = 140, d_p = 20$	2730
iv)	Header plate	$d_p = 175, t_p = 10, g = 74, t_w = 7.5$	2300