CIVIL ENGINEERING

Summary of IS 800 : 2007

Comprehensive Theory
with Solved Examples *and* Practice Questions

MADE EASY
PUBLICATIONS

www.madeeasypublications.org

MADE EASY Publications Pvt. Ltd.
Corporate Office: 44-A/4, Kalu Sarai (Near Hauz Khas Metro Station), New Delhi-110016 | **Ph. :** 9021300500
Email : infomep@madeeasy.in | **Web :** www.madeeasypublications.org

Summary of IS 800 : 2007

EDITIONS

First Edition: 2015
Second Edition: 2016
Third Edition: 2017
Fourth Edition: 2018
Fifth Edition: 2019
Sixth Edition: 2020
Seventh Edition: 2021
Eighth Edition: 2022

Ninth Edition: 2023

Summary of IS 800 : 2007

CONTENTS

SECTION 1 : GENERAL

1.1 SCOPE

1.1.1 This standard applies to general construction using hot rolled steel sections joined using riveting, bolting and welding. Specific provisions for bridges, chimneys, cranes, tanks, transmission line towers, bulk storage structures, tubular structures, cold formed light gauge steel sections, etc, are covered in separate standards.

1.3 TERMINOLOGY

1.3.3 **Action Effect or Load Effect:** The internal force, axial, shear, bending or twisting moment, due to external actions and temperature loads.

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

1.3.6 **Beam:** A member subjected predominately to bending.

1.3.13 **Camber:** Intentionally introduced pre-curving (usually upwards) in a system, member or any portion of a member with respect to its chord. Frequently, camber is introduced to compensate for deflections at a specific level of loads.

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

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

1.3.16 **Column:** A member in upright (vertical) position which supports a roof of floor system and predominantly subjected to compression.

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

1.3.30 **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.

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

1.3.33 **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.

1.3.35 **Effective Length:** Actual length of a member between points of effective restraint or effective restraint and free end, multiplied by a factor to take account of the end conditions in buckling strength calculations.

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

1.3.45 **Factor of Safety:** The factor by which the yield stress of the material of a member is divided to arrive at the permissible stress in the material.

1.3.46 **Fatigue:** Damage caused by repeated fluctuations of stress, leading to progressive cracking of a structural element.

1.3.51 **Fire Resistance:** The ability of an element, component or structure, to fulfil for a stated period of time, the required stability, integrity, thermal insulation and/or other expected performance specified in a standard fire test.

1.3.52 **Fire Resistive Level:** The fire resistance grading period for a structural element or system, in minutes, which is required to be attained in the standard fire test.

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

1.3.55 **Gauge:** The spacing between adjacent parallel lines of fasteners, transverse to the direction of load/stress.

1.3.57 **Gusset Plate:** The plate to which the members intersecting at a joint are connected.

1.3.63 **Limit State:** Any limiting condition beyond which the structure ceases to fulfil its intended function (see also **1.3.86**).

1.3.72 **Pitch:** The center-to-center distance between individual fasteners in a line, in the direction of load/stress.

1.3.73 **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.

1.3.86 **Serviceability Limit State:** A limit state of acceptable service condition exceedence of which causes serviceability failure.

1.3.88 **Shear Lag:** The in plane shear deformation effect by which concentrated forces tangential to the surface of plate gets distributed over the entire section perpendicular to the load over a finite length of the plate along the direction of the load.

1.3.91 **Slenderness Ratio:** The ratio of the effective length of a member to the radius of gyration of the cross-section about the axis under consideration.

1.3.93 **S-N Curve:** The curve defining the relationship between the number of stress cycles to failure (N_{SC}) at a constant stress range (S_c), during fatigue loading of a structure.

1.3.96 **Stability Limit State:** A limit state corresponding to the loss of static equilibrium of a structure by excessive deflection transverse to the direction of predominant loads.

1.3.102 **Strength Limit State:** A limit state of collapse or loss of structural integrity.

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

1.5 UNITS

For the purpose of design calculations the following units are recommended:

(a) Forces and loads, in kN, kN/m, kN/m^2
(b) Unit mass, in kg/m^3
(c) Unit weight, in kN/m^3
(d) Stresses and strengths in N/mm^2 (MN/m^2 or MPa)
(e) Moments (bending, etc) in kNm

1.8 CONVENTION FOR MEMBER AXES

Unless otherwise specified convention used for member axes is as follows.

(a) x-x along the member.
(b) y-y an axis of the cross-section.
  (i) perpendicular to the flanges, and
  (ii) perpendicular to the smaller leg in an angle section.
(c) z-z an axis of the cross-section.
  (i) axis parallel to flanges, and
  (ii) axis parallel to smaller leg in angle section.
(d) u-u major axis (when it does not coincide with z-z axis).
(e) v-v major axis (when it does not coincide with y-y aixs).

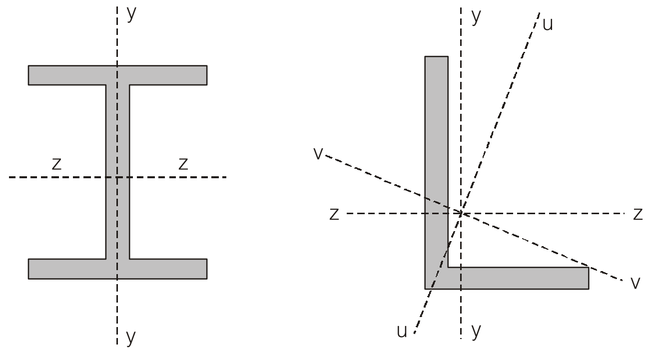


Fig. 1 Axes of Members

SECTION 2 : MATERIALS

2.1 GENERAL

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

2.2 STRUCTURAL STEEL

2.2.1 The provisions in this section are applicable to the steels commonly used in steel construction, namely, structural mild steel and high tensile structural steel.

2.4.1 Physical properties of structural steel irrespective of its grade may be taken as:

(a) Unit mass of steel, ρ = 7850 kg/m^3
(b) Modulus of elasticity, E = 2.0 × 10^5 N/mm^2 (MPa)
(c) Poission ratio, μ = 0.3
(d) Modulus of rigidity, G = 0.769 × 10^5 N/mm^2 (MPa)
(d) Coefficient of thermal expansion, α = 2 × 10^{-6}/°C

SECTION 3 : GENERAL DESIGN REQUIREMENTS

3.1 BASIS OF DESIGN

3.1.1 Design Objective

The objective of design is the achievement of an acceptable probability that structures will perform satisfactorily for the intended purpose during the design life. With an appropriate degree of safety, they should sustain all the loads and deformation, during construction and use and have adequate resistance to certain expected accidental loads and fire. Structure should be stable and have alternate load paths to prevent disproportionate overall collapse under accidental loading.

3.1.2 Methods of Design

3.1.2.1 Structure and its elements shall normally, be designed by the limit state method.

3.1.2.2 Where the limit states method cannot be conveniently adopted; the working stress design may be used.

3.2 LOADS AND FORCES

3.2.1 For the purpose of designing any element, member or a structure, the following loads (actions) and their effects shall be taken into account, where applicable, with partial safety factors and combinations (see **5.3.3**):

(a) Dead loads
(b) Imposed loads (live load, crane load, snow load, dust load, wave load, earth pressures, etc.)
(c) Wind loads
(d) Earthquake loads
(e) Erection loads
(f) Accidental loads such as those due to blast, impact of vehicles, etc.
(g) Secondary effects due to contraction or expansion resulting from temperature changes, differential settlements of the structure as a whole or of its components, eccentric connections, rigidity of joints differing from design assumptions.

3.3 ERECTION LOADS

All loads required to be carried by the structure or any part of it due to storage or positioning of construction material and erection equipment. Proper provision shall be made, including temporary bracings, to take care of all stresses developed during erection.

3.4 TEMPERATURE EFFECTS

3.4.1 Expansion and contraction due to changes in temperature of the members and elements of a structure shall be considered and adequate provision made for such effect.

3.5 LOAD COMBINATIONS

3.5.1 Load combinations for design purpose shall be those that produce maximum forces and effects and consequently maximum stresses and deformations. The following combination of loads with appropriate partial safety factors (see Table 4) may be considered:

(a) Dead load + imposed load
(b) Dead load + imposed load + wind or earthquake load
(c) Dead load + wind or earthquake load
(d) Dead load + erection load

3.5.2 Wind load and earthquake loads shall not be assumed to act simultaneously. The effect of each shall be considered separately.

3.5.7 Stresses developed due to secondary effects such as handling, erection, temperature and settlement of foundations, if any, shall be appropriately added to the stresses calculated from the combination of loads stated in **3.5.1**, with appropriate partial safety factors.

3.6 GEOMETRICAL PROPERTIES

3.6.1 General

The geometrical properties of the gross and the effective cross-sections of a member or part thereof, shall be calculated on the following basis:

(a) The properties of the gross cross-section shall be calculated from the specified size of the member or part thereof or read from appropriate table.

(b) The properties of the effective cross-section shall be calculated by deducting from the area of the gross cross-section, the following:

(i) The sectional area in excess of effective plate width, in case of slender sections (see **3.7.2**).

(ii) The sectional areas of the holes in the section except for parts in compression. In case of punched holes, hole size 2 mm in excess of the actual diameter may be deducted.

3.7 CLASSIFICATION OF CROSS-SECTIONS

3.7.1 Plate elements of a cross-section may buckle locally due to compressive stresses. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross-section subjected to compression due to axial force, moment or shear.

3.7.1.1 When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling, to enable the redistribution of bending moment required before formation of the failure mechanism.

3.7.2.1 When elastic analysis is used, the member shall be capable of developing the yield stress under compression without local buckling.

Example:

Q.1 In the case of structural steel sections, the MINIMUM ratio of thickness of elements in compression, in terms of their outstanding length is specified to prevent

(a) bending failure (b) shear failure
(c) local buckling (d) tension failure

[IES : 1998]

Ans. (c)

3.7.2 On basis of the above, four classes of sections are defined as follows:

(a) **Class-1 (Plastic):** Cross-sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism. The width to thickness ratio of plate elements shall be less than the limiting values.

(b) **Class-2 (Compact):** Cross-sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling. The width to thickness ratio of plate elements shall be less than the limiting value, but greater than that specified for Class-1 (Plastic).

(c) **Class-3 (Semi-compact):** Cross-section, in which the extreme fiber in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The width to thickness ratio of plate elements shall be less than the limiting value, but greater than that specified for Class-2 (Compact).

(d) **Class-4 (Slender):** Cross-sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate elements shall be greater than that specified for Class-3 (Semi-compact).

When different elements of a cross-section fall under different classes, the section shall be classified as governed by the most critical element.

3.7.3 Types of Elements

(a) **Internal elements:** These are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners, for example, web of I-section and flanges and web of box section.

(b) **Outside elements or outstands:** These are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane, for example flange overhang of an I-section, stem of T-section and legs of an angle section.

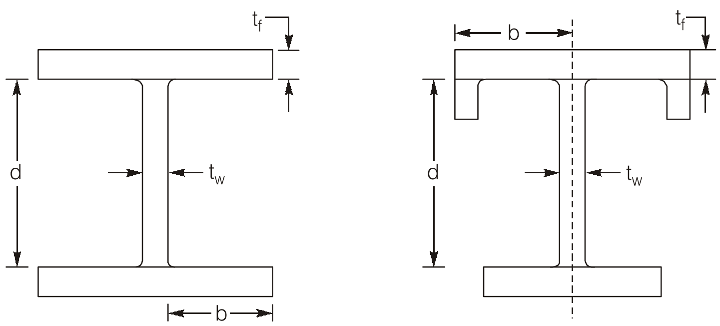
3.7.4 Compound Element in Built-up Section.

In case of compound elements consisting of two or more elements bolted or welded together, the limiting width to thickness ratios should be considered on basis of the following:

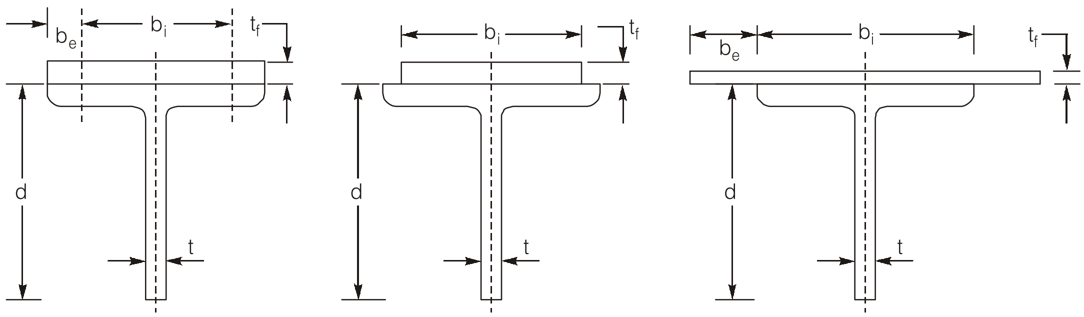
(a) Outstanding width of compound element (b_e) to its own thickness.

(b) The internal width of each added plate between the lines of welds or fasteners connecting it to the original section to own thickness.

(c) Any outstand of the added plates beyond the line of welds or fasteners connecting it to original section to its own thickness.



Built-up Sections



Compound Elements

b_i – Internal element width b_e – External element width

Fig. 2 Dimension of Sections

Example:

Q.1 The effective width of outstand in compound steel columns for design purposes is equal to
(a) half the flange width
(b) distance of the free edge from the rivet line
(c) distance of the free edge from the stiffeners
(d) distance of the free edge to the nearest row of rivets

[IES : 2013]

Ans. (d)

3.8 MAXIMUM EFFECTIVE SLENDERNESS RATIO

The maximum effective slenderness ratio, KL/r, values of a beam, strut or tension member shall not exceed those given in Table 3. 'KL' is the effective length of the member and 'r' is appropriate radius of gyration based on the effective section in **3.6.1**.

Table 3 Maximum Values of Effective Slenderness Ratios

S. No.	Member	Maximum Effective Slenderness Ratio (KL/r)
(I)	A member carrying compressive loads resulting from dead loads and imposed loads.	180
(II)	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces.	180
(III)	A member subjected to compression forces resulting only from combination with wind/earthquake actions, provided the deformation of such member does not adversely affect the stress in any part of the structure.	250
(IV)	Compression flange of a beam against lateral torsional buckling.	300
(V)	A members normally acting as a tie in a roof truss or a bracing system and considered effective when subject to possible reversal of stress into compression resulting from the action of wind or earthquake forces.	350
(VI)	Members always under tension (other than pre-tensioned members).	400
Note: Tension members, such as bracings, pre-tensioned to avoid sag, need not satisfy the maximum slender-ness ratio limits.		

Example:

Q.1 The slenderness ratio in tension member as per BIS code where reversal of stress is due to loads other than wind or seismic shall not exceed

(a) 350
(b) 180
(c) 100
(d) 60

[IES : 2001]

Ans. (b)

Q.2 A steel rod of 16 mm diameter has been used as a tie in a bracing system, but may be subject to possible reversal of stress due to the wind. What is the maximum permitted length of the member?

(a) 1600 mm
(b) 1400 mm
(c) 1200 mm
(d) 1000 mm

[IES : 2005]

Ans. (b)

Radius of gyration of steel rod, r

$$= \frac{D}{4} = \frac{16}{4} = 4 \text{ mm}$$

Tie is a tension member.

For reversal of stress, the maximum permissible slenderness ratio = 350

∴ Length of member permitted = 350 × 4

= 1400 mm

Q.3 **Assertion (A):** Slenderness ratio of tension members is restricted to 250.

Reason (R): Slenderness ratio for tension members is a stiffness criterion associated with self weight.

(a) both A and R are individually true and R is the correct explanation of A
(b) both A and R are individually true but R is not the correct explanation of A
(c) A is true but R is false
(d) A is false but R is true

[IES : 2007]

Ans. (d)

Q.4 M60 structural steel tube has a radius of gyration 20 mm. The unbraced length upto which the tube can be used as a compression member, is

(a) 3.6 m
(b) 5.0 m
(c) 6.0 m
(d) 7.2 m

[IES : 2001]

Ans. (a)

Maximum permissible slenderness ratio = 180

∴ Unbraced length = 180 × 20

= 3600 mm = 3.6 m

Q.5 Match **List-I** (Type of member) with **List-II** (Slenderness ratio) and select the correct answer using the codes given below the lists:

List-I

A. For compression members carrying dead and superimposed loads
B. For members carrying compressive loads due to wind or seismic forces only
C. For members carrying tension but in which the reversal of stress occurs due to wind or seismic forces

List-II
1. 350
2. 180
3. 250

Codes:

	A	B	C
(a)	1	2	3
(b)	2	3	1
(c)	3	1	2
(d)	1	3	2

[IES : 2002]

Ans. (b)

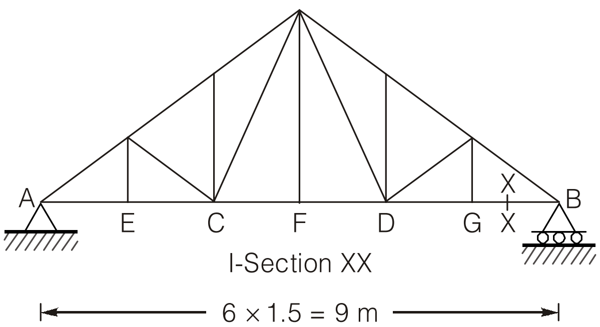
Q.6 An electric pole 5 m high is fixed into the foundation. It carries a wire at the top and is free to move sideways. The effective length of the pole is

(a) 3.25 m
(b) 4.0 m
(c) 5.0 m
(d) 10.0 m

[IES : 2003]

Ans. (b)

Q.7 For the roof truss shown in figure below, bottom chord is of ISMB 200 (r_x = 83 mm, r_y = 22 mm)



Bottom chord bracings are available at C and D. Bottom member AE will be in compression due to wind. What is the critical slenderness ratio used for the design of member AE?

(a) 18
(b) 36
(c) 68
(d) 136

[IES : 2010]

Ans. (c)

Slenderness ratio is given by

$$l = \frac{L_{eff}}{r_{min}} = \frac{1.5 \times 10^3}{22} = 68$$

3.9 RESISTANCE TO HORIZONTAL FORCES

3.9.1 In designing the steel frame work of a building, provision shall be made (by adequate moment connections or by a system of bracing) to effectively transmit to the foundations all the horizontal forces, giving due allowance for the stiffening effect of the walls and floors, where applicable.

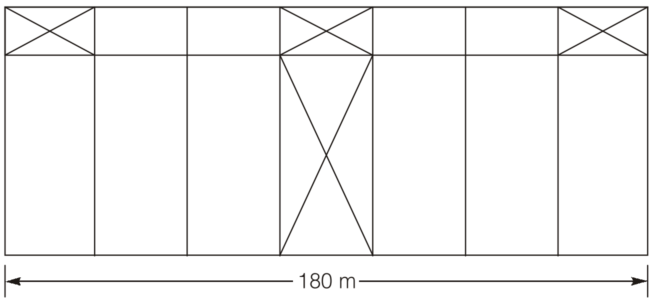
3.9.2 When the walls, or walls and floors and/or roofs are capable of effective transmitting all the horizontal forces directly to the foundations, the structural steel framework may be designed without considering the effect of wind or earthquake.

3.10.2 Structures in which marked changes in plan dimensions take place abruptly, shall be provided with expansion joints at the section where such changes occur. Expansion joints shall be so provided that the necessary movement occurs with minimum resistance at the joint. The gap at the expansion joint should be such that:

(a) It accommodates the expected expansion/contraction due to seasonal and diurnal variation of temperature, and

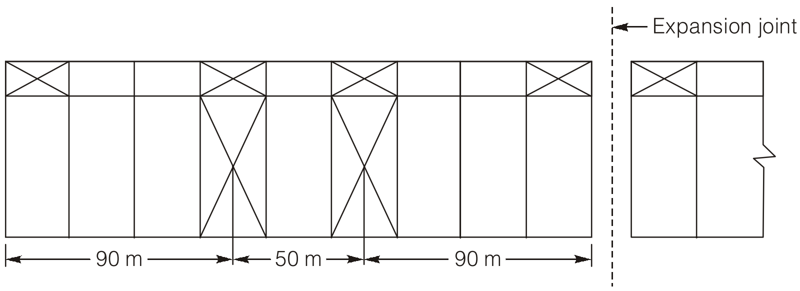
(b) It avoids pounding of adjacent units under earthquake. The structure adjacent to the joint should preferably be supported on separate columns but not necessarily on separate foundations.

3.10.3.1 If one bay of longitudinal bracing is provided at the center of the building or building section, the length of the building section may be restricted to 180 m in case of covered buildings and 120 m in case of open gantries.



(End of covered building/section)
Maximum Length of Building with one Bay of Bracing

3.10.3.2 If more than one bay of longitudinal bracing is provided near the center of the building/ section, the maximum center line distance between the two lines buildings (and 30 m for open gantries) and the maximum distance between the center of the bracing to the nearest expansion joint/end of building or section may be restricted to 90 m (60 m in case of open gantries). The maximum length of the building section thus may be restricted to 230 m for covered buildings (150 m for open gantries). Beyond this, suitable expansion joints shall be provided.



Maximum Length of Building/Section with two Bays of Bracing

3.10.3.3 The maximum width of the covered building section should preferably be restricted to 150 m beyond which suitable provisions for the expansion joint may be made.

3.10.4 When the provisions of these sections are met for a building or open structure, the stress analysis due to temperature is not required.

SECTION 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 B, and
(d) Dynamic analysis in accordance with IS 1893 (Part I)

4.1.2 Non-sway and Sway Frames

For the purpose of analysis and design, the structural frames are classified as non-sway and sway frames as given below:

(a) **Non-sway frame:** One in which the transverse displacement of one end of the member relative to the other end is effectively prevented. This applies to triangulated frames and trusses or to frames where in-plane stiffness is provided by bracings, or by shear walls, or by floor slabs and roof decks secured horizontally to walls or to bracing systems parallel to the plane of loading and bending of the frame.

(b) **Sway frame:** One in which the transverse displacement of one end of the member relative to the other end is not effectively prevented. Such members and frames occur in structures which depend on flexural action of members to resist lateral loads and sway, as in moment resisting frames.

(c) A rigid jointed multi-storey frame may be considered as a non-sway frame if in every individual storey, the deflection δ, over a storey height h_s, due to the notional horizontal loading (given in **4.3.6**) satisfies the following criteria:

1. For clad frames, when the stiffening effect of the cladding is not taken into account in the deflection calculations:

$$\delta \leq \frac{h_s}{2000}$$

2. For unclad frame or clad frames, when the stiffening effect on the cladding is taken into account in the deflection calculations:

$$\delta \leq \frac{h_s}{4000}$$

3. A frame, which when analyzed considering all the lateral supporting system does not comply with the above criteria, should be classified as a sway frame, even if it is braced or otherwise laterally stiffened.

4.2 FORMS OF CONSTRUCTION ASSUMED FOR STRUCTURAL ANALYSIS

4.2.1.1 Rigid construction

In rigid construction, the connections between members (beam and column) at their junctions shall be assumed to have sufficient rigidity to hold the original angles between the members connected at a joint unchanged under loading.

4.2.1.2 Semi-rigid construction

In semi-rigid construction, the connections between members (beam and column) at their junction may not have sufficient rigidity to hold the original angles between the members at a joint unchanged, but shall be assumed to have the capacity to furnish a dependable and known degree of flexural restraint. The relationship between the degree of flexural restraint and the level of the load effects shall be established by any rational method or based on test results (see Annex F).

4.2.1.3 Simple construction

In simple construction, the connections between members (beam and column) at their junction will not resist any appreciable moment and shall be assumed to be hinged.

4.3 ASSUMPTIONS IN ANALYSIS

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

(a) Regular building structures, with orthogonal frames in plane, may be analysed as a series of parallel two-dimensional sub-structures (part of a structure), the analysis being carried out in each of the two directions, at right angles to each other. For earthquake loading three dimensional analysis may be necessary to account for effects of torsion and also for multi-component earthquake forces.

(b) For vertical loading in a multi-storey 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.

(c) 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.

4.2.1.3 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.

4.2.1.3 Arrangements of Imposed Loads in Buildings

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

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

(b) Where the imposed 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 imposed 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. Imposed load on alternate spans.
   2. Imposed load on two adjacent spans, and
   3. Imposed load on all the spans.

4.3.4 Base Stiffness

In the analysis of all structures the appropriate base stiffness about the axis under consideration shall be used. In the absence of the knowledge of the pedestal and foundation stiffness, the following may be assumed:

(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 the 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:
   1. When the foundation consist 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.
   2. When the foundation consists of an isolated footing resting on other soils, pedestal shall be assumed to hinged at the level of the bottom of footing.
   3. 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.
   4. 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.

4.3.6 Notional Horizontal Loads

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

4.3.6.1 The notional forces should be applied on the whole structure, in both orthogonal directions, in one direction at a time, at roof and all floor levels or their equivalent. They should be taken as acting simultaneously with factored gravity loads.

4.3.6.2 The notional force should not be
(a) applied when considering overturning or overall instability;
(b) combined with other horizontal (lateral) loads;
(c) combined with temperature effects; and
(d) taken to contribute to the net shear on the foundation.

4.4 ELASTIC ANALYSIS

4.4.1 Assumptions

Individuals members shall be assumed to remain elastic under the action of the factored design loads for all limit states.

The effect of launching 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 underformed geometry is considered, the changes in the geometry of the frame due to the loading are not accounted for, and changes in the effective stiffness of the members due to axial force are neglected.

4.4.3.4 The calculated bending moments from the first order elastic analysis may be modified by redistribution upto 15 percent 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 shall belong to plastic or compact section classification.

4.5 PLASTIC ANALYSIS

4.5.2 Requirements

When a plastic method of analysis is used all of the following conditions shall be satisfied.
(a) The yield stress of the grade of the steel used shall not exceed 450 MPa.
(b) The stress-strain characteristics of the steel shall be such as to ensure complete plastic moment redistribution. The stress-strain diagram shall have a plateau at the yield stress, extending for at least six times the yield strain. The ratio of the tensile strength to the yield stress specified for the grade of the yield stress specified for the grade of the steel shall not be less than 1.2. The elongation on a gauge length shall not be less than 15 percent, and the steel shall exhibit strain-hardening capability.
(c) The members used shall be hot-rolled or fabricated using hot-rolled plates and sections.
(d) The cross-section of members not containing plastic hinges should be at least that of compact section, unless the members meet the strength requirements from elastic analysis.

(e) Where plastic hinges occurs in a member, the proportions of its cross-section should not exceed the limiting values for the plastic section given in **3.7.2**.

(f) The cross-section should 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) should be provided at all plastic hinge locations. Where not feasible, the restraint should 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 meet at the last plastic hinge to form.

Within a member containing a plastic hinge, the maximum distance L_m from the restraint at the plastic hinge to an adjacent restraint should 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

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

where, f_c = actual 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, $x_t = 1.132\,(AI_w/I_yI_t)^{0.5}$;

A = area of cross-section; and

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 or the values of the compression flanges only or the whole section.

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

The spacing of restraints to member lengths not containing a plastic hinge should satisfy the recommendations of section on lateral buckling strength of beams. (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 should be provided where a concentrated load, which exceeds 10 percent of the shear capacity of the member, is applied within D/2 of a plastic hinge location. The stiffener should be provided within a distance of half the depth of the member on either side of the hinge location.

4.5.2.3 The frame shall be adequately supported against sway and out-of-plane buckling, by bracings, moment resisting frame or an independent system such as shear wall.

4.5.2.4 Fabrication Restriction

Within a length equal to the member depth, on either side of a plastic hinge location, the following restrictions should be applied to the tension flange and noted in the design drawings. Holes if required, should be drilled or else punched 2 mm undersize and reamed. All sheared or hand flame cut edges should 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, the moment capacity of the connection may be less than that of the member being connected; and

(c) in both cases the behaviour 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.

SECTION 5 : LIMIT STATE DESIGN

5.1 BASIS OF DESIGN

5.1.1 In the limit state design method, the structure shall be designed to withstand safely all loads likely to act on it throughout its life. It shall not suffer total collapse under accidental loads such as from explosions or impact or due to consequences of human error to an extent beyond the local damages. The objective of the design is to achieve a structure that will remain fit for use during its life with acceptable target reliability. In other words, the probability of a limit state being reached during its lifetime should be very low. The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

5.1.2 Steel structure are to be designed and constructed to satisfy the design requirements with regard to stability, strength, serviceability, brittle fracture, fatigue, fire and durability such that they meet the following:

(a) Remain fit with adequate reliability and be able to sustain all actions (loads) and other influence experienced during construction and use

(b) Have adequate durability under normal maintenance

(c) Do not suffer overall damage or collapse disproportionately under accident events like explosions, vehicle impact or due to consequences of human error to an extent beyond local damage.

The following conditions may be satisfied to avoid a disproportionate collapse:

(a) The building should be effectively tied together at each principal floor level and each column should be effectively held in position by means of continuous ties (beams) nearly orthogonal.

5.2 LIMIT STATE DESIGN

5.2.1 For achieving the design objectives, the design shall be based on characteristic values for material strengths and applied loads (actions), which take into account the probability of variations in the material strengths and in the loads to be supported. The reliability of design is ensured by satisfying the requirement:

$$\text{Design action} \leq \text{Design strength}$$

5.2.2 Limit states are the states beyond which the structure no longer satisfies the performance requirements specified. The limit states are classified as:

(a) Limit state of strength
(b) Limit state of serviceability

5.2.2.1 The limit states of strength are those associated with failures (or imminent failure), under the action of probable and most unfavourable combination of loads on the structure using the appropriate partial safety factors, which may endanger the safety of life and property. The limit state of strength includes:

(a) Loss of equilibrium of the structure as a whole or any of its parts of components.
(b) Loss of stability of the structure (including the effect of sway where appropriate and overturning) or any of its parts including supports and foundations.
(c) Failure by excessive deformation, rupture of the structure or any of its parts of components.
(d) Fracture due to fatigue.
(e) Brittle fracture.

5.2.2.2 The limit state of serviceability includes:

(a) Deformation and deflections, which may adversely affect the appearance or effective use of the structure or may cause improper functioning of equipment or services or may cause damages to finishes and non-structural members
(b) Vibrations in the structure or any of its components causing discomfort to people, damages to the structure, its contents or which may limit its functional effectiveness. Special consideration shall be given to systems susceptible to vibration, such as large open floor areas free of partitions to ensure that such vibrations are acceptable for the intended use and occupancy
(c) Repairable damage or crack due to fatigue
(d) Corrosion, durability
(e) Fire

5.3 ACTIONS

The actions (loads) to be considered in design include direct actions (loads) experienced by the structure due to self weight, external actions etc., and imposed deformations such as that due to temperature and settlements.

5.3.1 Classification of Actions

Actions are classified by their variation with time as given below:

(a) **Permanent action (Q):** Actions due to self-weight of structural and non-structural components, fittings, ancillaries, and fixed equipment, etc.

(b) **Variable action (Q):** Actions due to construction and service stage loads such as imposed (live) loads (crane loads, snow loads, etc.), wind loads, and earthquake loads, etc. to explosions, and impact of vehicles, etc.

5.3.2 Characteristic Actions (Loads)

5.3.2.1 The Characteristic Action, Q_c are the values of the different actions that are not expected to be exceeded with more than 5 percent probability, during the life of the structure.

5.3.3 Design Actions

The Design Action, Q_d is expressed as $Q_d = \sum_k \gamma_{fk} Q_{ck}$

where, γ_{fk} = partial safety factor for different loads k, given in Table 4 to account for:

(a) Possibility of unfavourable deviation of the load from the characteristic value,

(b) Possibility of inaccurate assessment of effects of the load, and

(c) Uncertainty in the assessment of effects of the load, and

(d) Uncertainty in the assessment of the limit states being considered.

The loads or load effects shall be multiplied by the relevant γ_f factors, given in Table 4, to get the design loads or design load effects.

Table 4 Partial Safety Factors for Loads, γ_f for Limit States

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL		WL/EL	AL	DL	LL		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL + LL + CL	1.5	1.5	1.05	–	–	1.0	1.0	1.0	–
DL + LL + CL$^+$	1.2	1.2	1.05	0.6	–	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2					
DL + WL/EL	1.5	–	–	1.5	–	1.0	–	–	1.0
DL + ER	1.2	1.2	–	–	–	–	–	–	–
DL + LL + AL	1.0	0.35	0.35	–	1.0	–	–	–	–

Abbreviations:
DL = Dead load, LL = Imposed load (Live load), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.

5.4 STRENGTH

The ultimate strength calculation may require consideration of the following:

(a) Loss of equilibrium of the structure or any part of it, considered as a rigid body; and

(b) Failure by excessive deformation, rupture or loss of stability of the structure or any part of in including support and foundation.

5.4.1 Design Strength

The Design Strength, S_d is obtained as given below from ultimate strength, S_u and partial safety factors for materials, γ_m given in Table 5.

$$S_d = S_u / \gamma_m$$

where partial safety factor for materials, γ_m account for:

(a) Possibility of unfavourable deviation of material strength from the characteristic value,

(b) Possibility of unfavourable variation of member sizes,

(c) Possibility of unfavourable reduction in member strength due to fabrication and tolerances, and

(d) Uncertainty in the calculation of strength of the members.

Table 5 Partial Safety Factor for Materials, γ_m

Sl. No.	Definition	Partial safety factor	
(i)	Resistance, governed by yielding, γ_{m0}	1.10	
(ii)	Resistance of member to buckling, γ_{m0}	1.10	
(iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
(iv)	Resistance of connection:	Shop fabrications	Field fabrications
	(a) Bolts-friction type, γ_{mf}	1.25	1.25
	(b) Bolts-bearing type, γ_{mb}	1.25	1.25
	(c) Rivets, γ_{mr}	1.25	1.25
	(d) Welds, γ_{mw}	1.25	1.50

5.5 FACTORS GOVERNING THE ULTIMATE STRENGTH

5.5.1 Stability

Stability shall be ensured for the structure as a whole and for each of its elements. This should include, overall frame stability against overturning and sway, as given in **5.5.1.1** and **5.5.1.2**.

5.5.1.1 Stability Against Overturning

The structure as a whole or any part of it shall be designed to prevent instability due to overturning, uplift or sliding under factored load as given below:

(a) The Actions shall be divided into components aiding instability and components resisting instability.

(b) The permanent and variable actions and their effect causing instability shall be combined using appropriate load factors as per the Limit State requirements, to obtain maximum destabilizing effect.

(c) The permanent actions (loads) and effects contributing to resistance shall be multiplied with a partial safety factor 0.9 and added together with design resistance (after multiplying with appropriate partial safety factor). Variable actions and their effects contributing to resistance shall be disregarded.

(d) The resistance effect shall be greater than or equal to the destabilizing effect. Combination of imposed and dead loads should be such as to cause most severe effect on overall stability.

5.5.1.2 Sway Stability

The whole structure, including portions between expansion joints, shall be adequately stiff against sway. To ensure this, in addition to designing for applied horizontal loads, a separate check should be carried out for notional horizontal loads such as given in 4.3.6 to evaluate the sway under gravity loads.

5.5.2 Fatigue

Generally fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to fluctuations in wind loading normally need not be considered. Fatigue design shall be in accordance with Section 13. When designing for fatigue, the partial safety factor for load, γ_f equal to unity shall be used for the load causing stress fluctuation and stress range.

5.5.3 Plastic Collapse

Plastic analysis and design may be used, if the requirement specified under the plastic method of analysis are satisfied.

5.6 LIMIT STATE OF SERVICEABILITY

Serviceability limit state is limit state beyond which the service criteria specified below, are no longer met:

(a) Deflection limit
(b) Vibration limit
(c) Durability consideration
(d) Fire resistance

Unless specified otherwise, partial safety factor for loads, γ_f of value equal to unity shall be used for all loads leading to serviceability limit states to check the adequacy of the structure under serviceability limit states.

5.6.1 Deflection

The deflection under serviceability loads of a building or a building components should not impair the strength of the structure or components or cause damage to finishing. Deflection are to be checked for the most adverse but realistic combination of service loads and their arrangement, by elastic analysis, using a load factor of 1.0.

5.6.2 Vibration

Suitable provisions in the design shall be made for the dynamic effects of live loads, impact loads and vibration due to machinery operating loads. In severe cases possibility of resonance, fatigue or unacceptable vibrations shall be investigated. Unusually flexible structures (generally the height to effective width of lateral load resistance system exceeding 5 : 1) shall be investigated for lateral vibration under dynamic wind loads. Structures subjected to large number of cycles of loading shall be designed against fatigue failure.

5.6.3 Durability

Factors that affect the durability of the buildings, under conditions relevant to their intended life, are listed below:

(a) Environment,
(b) Degree of exposure
(c) Shape of the member and the structural detail,
(d) Protective measure, and
(e) Ease of maintenance.

5.6.4 Fire Resistance

Fire resistance of a steel member is a function of its mass, its geometry, the actions to which it is subjected, its structural support condition, fire protection measures adopted and the fire to which it is exposed.

SECTION 6 : DESIGN OF TENSION MEMBERS

6.1 TENSION MEMBERS

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads upto the ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions (see 6.4.1).

The factored design tension T, in the members shall satisfy the following requirements:

$$T < T_d$$

where, T_d = design strength of the member.

The design strength of a member under axial tension, T_d is the lowest of the design strength due to yielding of gross-section, T_{dg}; rupture strength of critical section, T_{dn}; and block shear T_{db}; given in **6.2**, **6.3** and **6.4**, respectively.

6.2 DESIGN STRENGTH DUE TO YIELDING OF GROSS-SECTION

The design strength of members under axial tension, T_{dg}, as governed by yielding of gross-section, is given by

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

where, f_y = yield stress of the material,
A_g = gross area of cross-section, and
λ_{m0} = partial safety factor for failure in tension by yielding (see Table 5).

6.3 DESIGN STRENGTH DUE TO RUPTURE OF CRITICAL SECTION

6.3.1 Plates

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

$$T_{dn} = \frac{0.9\, A_n f_u}{\gamma_{m1}}$$

where, γ_{m1} = partial safety factor for failure at ultimate stress (see Table),
f_u = ultimate stress of the material, and
A_n = net effective area of the member given by,

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

where, b, t = width and thickness of the plate, respectively
d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes)
g = gauge length between the bolt holes, as shown in figure
p_s = staggered pitch length between line of bolt holes, as shown in figure
n = number of bolt holes in the critical section
i = subscript for summation of all the inclined legs

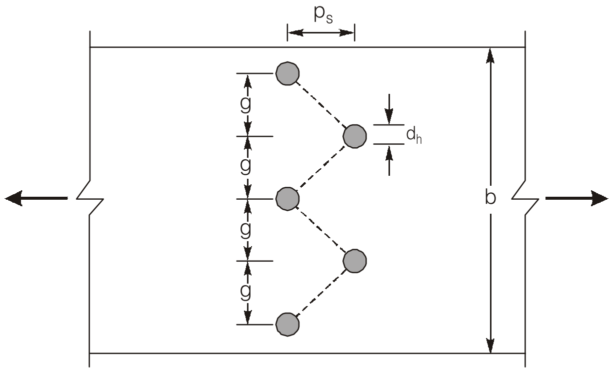
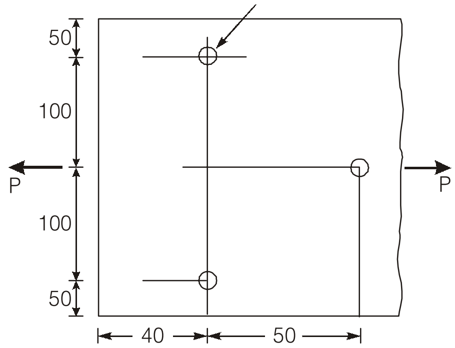


Fig. 5 Plates with bolts holes in tension

Example:

Q.1 What is the effective net width of plate shown in the given sketch, for carrying tension?



(a) 212.5 mm (b) 237.5 mm
(c) 250 mm (d) 275 mm

[IES : 1996]

Ans. (b)

Effective net width of plate

$$= 300 - \left[3d - \frac{s_1^2}{4g_1} - \frac{s_2^2}{4g_2}\right]$$

$$s_2 = s_1 = 50 \text{ mm}$$
$$g_1 = g_2 = 100 \text{ mm}$$
$$d = 25 \text{ mm}$$

$$\therefore \quad \text{Net width} = 300 - \left[3 \times 25 - \frac{2 \times 50^2}{4 \times 100}\right]$$

$$= 237.5 \text{ mm}$$

6.3.2 Threaded Rods

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

$$T_{dn} = \frac{0.9\, A_n\, f_u}{\gamma_{m1}}$$

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

6.3.3 Single Angles

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn}, as governed by rupture at net section is given by:

$$T_{dn} = \frac{0.9\, A_{nc}\, f_u}{\gamma_{m1}} + \frac{\beta\, A_{go} f_y}{\gamma_{m0}}$$

where, $\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \le (f_u\,\gamma_{m0}/f_y\,\gamma_{m1}) \ge 0.7$

w = outstand width,

b_s = shear lag width, as shown in Fig. 6, and

L_e = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

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

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

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 the connected leg;

A_{go} = gross area of the outstanding leg; and

t = thickness of the leg

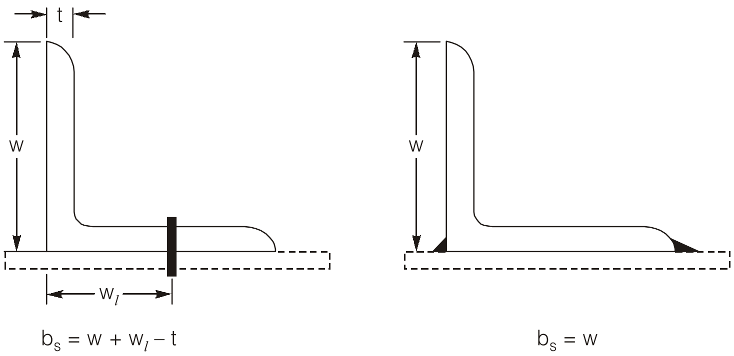


Fig. 6 Angles with single leg connections

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 in **6.4.1**.

6.4.1 Bolted Connections

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

$$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right]$$

or

$$T_{db} = \left(\frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right)$$

where, A_{vg}, A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively (1-2 and 3-4 as shown in Fig. (7A) and 1 – 2 as shown in Fig. (7B)).

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively (2-3 as shown in Fig. (7B)), and

f_u, f_y = ultimate and yield stress of the material, respectively.

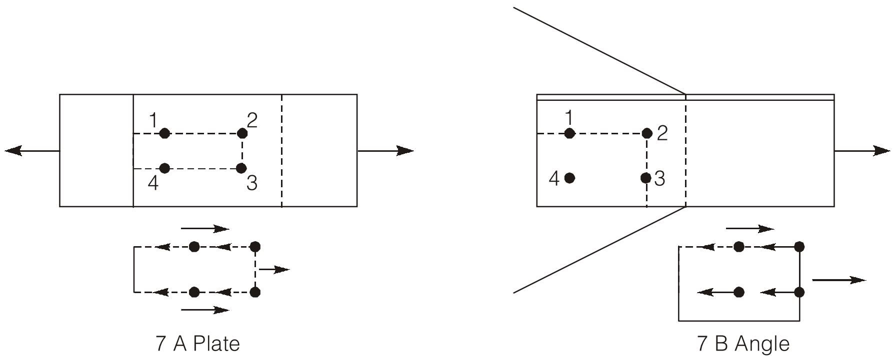


Fig. 7 Block shear failure

6.4.2 Welded Connection

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.

SECTION 7 : DESIGN OF COMPRESSION MEMBERS

7.1 DESIGN STRENGTH

7.1.1 Commonly hot rolled and built-up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a, b, c or d as given Table 7.

7.1.2 The design compressive strength P_d, of a member is given by:

$$P < P_d$$

where, $P_d = A_e f_{cd}$

where, A_e = effective sectional area as defined in **7.3.2** and

f_{cd} = design compressive stress, obtained as per **7.1.2.1**.

7.1.2.1 The design compressive stress, f_{cd}, of axially loaded compression members shall be calculated using the following equation:

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

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

λ = non-dimensional effective slenderness ratio

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

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

where, KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r;

α = imperfection factor given in Table 7;

χ = stress reduction factor for different buckling class, slenderness ratio and yield stress

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

λ_{m0} = partial safety factor for material strength.

7.1.2.2 The curve corresponding to different buckling class are presented in non-dimensional form, Fig. 8.

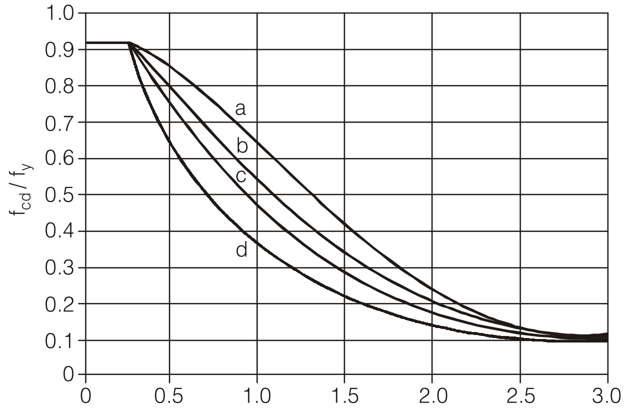


Fig. 8 Column buckling curves

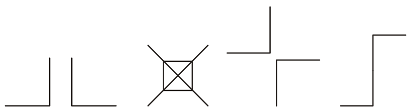
Table 7 Imperfection Factor, α

(Clauses 7.1.1 and 7.1.2.1)

Buckling class	a	b	c	d
α	0.21	0.34	0.49	0.76

Example:

Q.1 Two equal angles form a compound column cross-section as shown in figures 1, 2, 3 and 4.



Among these, those which have the same axial compression load carrying capacity would include

(a) 1 and 2 (b) 1 and 3 (c) 2 and 3 (d) 3 and 4

[IES : 1997]

Ans. (c)

The cruciform arrangement of double angle, in star section (figure-3) is most effective because of its approximately equal radii of gyration in two directions.

The box section (figure-2) can be formed with welded connection. The least radius of gyration of the section shown in the figure-2 and figure-3 of two angles is same and it is more than that of the sections shown in figure-1. Therefore the permissible compressive stress for figure 2 and 3 will be same and compression load carrying capacity will be same.

Q.2 Which of the following parameters govern the permissible stress in compression in columns?

1. Modulus of section 2. Effective length 3. Radius of gyration

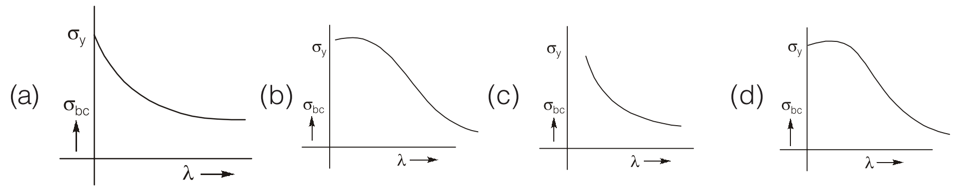
Select the correct answer using the codes given below:

(a) 1, 2 and 3 (b) 1 and 2 only (c) 2 and 3 only (d) 1 and 3 only

[IES : 2008]

Ans. (a)

Q.3 Which one of the following graphs represents the compressive strength (s_{bc}) versus slenderness ratio (λ)?



[IES : 2009]

Ans. (c)

Q.4 The most critical consideration in the design of a rolled steel column carrying axial loads is the
(a) Percentage elongation at yield and the net cross-sectional area
(b) Critical bending strength and axial yield strength of material
(c) Buckling strength based on the net area of the section and percentage elongation at ultimate load
(d) Compressive strength based on slenderness ratio and gross cross-sectional area

[IES : 2011]

Ans. (d)

Q.5 Consider the following parameters with regards to slenderness ratio of a compression member:
1. Material
2. Sectional configuration
3. Length of member
4. Support end conditions

On which of these parameters does the slenderness ratio of a compression member depend?
(a) 1, 2 and 3 only (b) 1, 3 and 4 only (c) 2, 3 and 4 only (d) 1, 2, 3 and 4

[IES : 2012]

Ans. (c)

7.2 EFFECTIVE LENGTH OF COMPRESSION MEMBERS

The effective length KL, is calculated from the actual length L, of the member, considering the rotational and relative translational boundary conditions at the ends. The actual length shall be taken as the length from center-to-center of its intersections with the supporting members in the plane of the buckling deformation. In the case of a member with a free end, the free standing length from the center of the intersecting member at the supported end, shall be taken as the actual length.

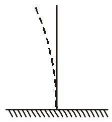
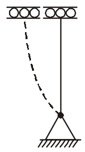
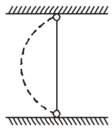
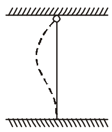
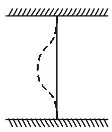
7.2.2 Effective Length

Where the boundary conditions in the plane of buckling can be assessed, the effective length, KL can be calculated on the basis of Table 11.

7.2.4 Compression Members in Trusses

In the case of bolted, riveted or welded trusses and braced frames, the effective length, KL, of the compression members shall be taken as 0.7 to 1.0 times the distance between centers of connections, depending on the degree of end restraint provided. In the plane perpendicular to the plane of the truss, the effective length KL shall be taken as the distance between the centers of intersection. The design of angle struts shall be as specified in 7.5.

Table 11 Effective Length of Prismatic Compression Members

Boundary Condition				Schematic Representation	Effective Length
At one end		At the other end			
Translation (1)	Rotation (2)	Translation (3)	Rotation (4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Free		0.65L

Note: L is the unsupported length of the compression member.

Example:

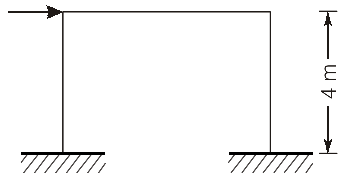
Q.1 The effective length of a structural steel compression member of length L effectively held in position and restrained against rotation at one end but neither held in position nor restrained against rotation at the other end, is

(a) L (b) 1.2L
(c) 1.5L (d) 2.0L

[IES : 1998]

Ans. (d)

Q.2 An industrial portal frame shown has weak floor beams. What is the effective length of column?



(a) 3 m (b) 4 m
(c) 6 m (d) 8 m

[IES : 2006]

Ans. (d)
Since the floor beam is weak so the top end of the column behaves like a free end. Therefore the effective length of the column is $2 \times 4 = 8$ m.

Q.3 A steel column pinned at both ends has a buckling load of 200 kN. If the column is restrained against lateral movement at its mid-height, its buckling load will be
(a) 200 kN (b) 283 kN
(c) 400 kN (d) 800 kN

[IES : 2012]

Ans. (d)

7.3 DESIGN DETAILS

7.3.1 Thickness of Plate Elements

Classification of members on the basis of thickness of constituent plate elements shall satisfy the width-thickness ratio requirements.

Example:

Q.1 In a compression member, plate element may buckle locally before the entire member fails. To avoid this, which of the following recommendations are made?
1. Thickness of members is taken in terms of lengths of compression members.
2. Length of element is increased
3. Length of member is increased
4. Length of outstand is decreased

Select the correct answer using the codes given below:
(a) 1, 2 and 3 (b) 1, 3 and 4
(c) 2 and 3 (d) 1 and 4

[IES : 2002]

Ans. (d)

7.3.2 Effective Sectional Area, A_e

Except as modified in **3.7.2** (Class 4), the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding, bolting and riveting so long as the section is semi-compact or better. Holes not fitted with rivets, bolts or pins shall be deducted from gross area to calculated effective sectional area.

7.3.3 Eccentricity for Stanchions and Columns

7.3.3.1 For the purpose of determining the stress in a stanchion or column section, the beam reactions or similar loads shall be assumed to be applied at an eccentricity of 100 mm from the face of the section or at the center of bearing whichever dimension gives the greater eccentricity, and with the exception of the following two cases:

(a) In the case of cap connection, the load shall be assumed to be applied at the face of the column or stanchion section or at the edge of packing, if used towards the span of the beam.

(b) In the case of a roof truss bearing on a cap, no eccentricity be taken for simple bearings without connections capable of developing any appreciable moment. In case of web member connection with face, actual eccentricity is to be considered.

7.3.3.2 In continuous columns, the bending moments due to eccentricities of loading on the columns at any floor may be divided equally between the columns above the below that floor level, provided that 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. Where 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 Where the ends of compression members are prepared for bearing over the whole area, they shall be spliced to hold the connected members accurately in position, and to resist bending or tension, if present. Such splices should maintain the intended member stiffness about each axis. Splices should be located as close to the point of inflection as possible. Otherwise their capacity should be adequate to carry magnified moment. The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing.

7.3.4.2 Where such members are not faced for complete bearing, the splices shall be designed to transmit all the forces to which the members are subjected.

7.3.4.3 Wherever possible, splices shall be proportional and arranged so that the centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members being jointed, in order to avoid eccentricity, but where eccentricity is present in the joint, the resulting stress shall be accounted for.

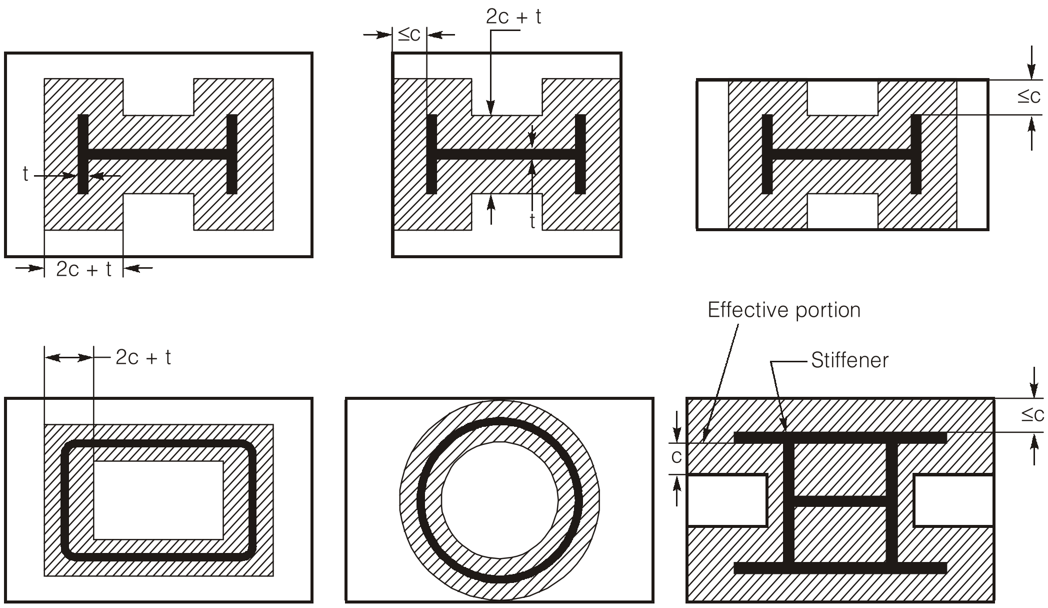
7.4 COLUMN BASES

7.4.1 General

Column bases should have sufficient stiffness and strength to transmit axial force, bending moments and shear forces at the base of the columns to their foundation without exceeding the load carrying capacity of the supports. Anchor bolts and shear keys should 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.

The nominal bearing pressure between the base plate and the support below may be determined on the basis of linearly varying distribution of pressure. The maximum bearing pressure should not exceed the bearing strength equal to 0.6 f_{ck}, where f_{ck} is the smaller of characteristic cube strength of concrete or bedding material.

7.4.1.1 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 Fig. 9, such that bearing pressure on the effective area does not exceed bearing capacity of concrete base.



Effective area of a base plate

7.4.2 Gusseted Bases

For stanchion with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc, in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces shall be machined to ensure perfect contact.

7.4.2.1 Where the ends of the column shaft and the gusset plates are not faced for complete bearing, the welding, fastening connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

7.4.2.2 Column and Base Plate Connections

Where the end of the column is connected directly to the base plate by means of full penetration butt welds, the connection shall be deemed to transmit to the base all the forces and moments to which the column is subjected.

7.4.3 Slab Bases

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

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

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

where, w = uniform pressure from below on the slab base under the factored load axial compression;

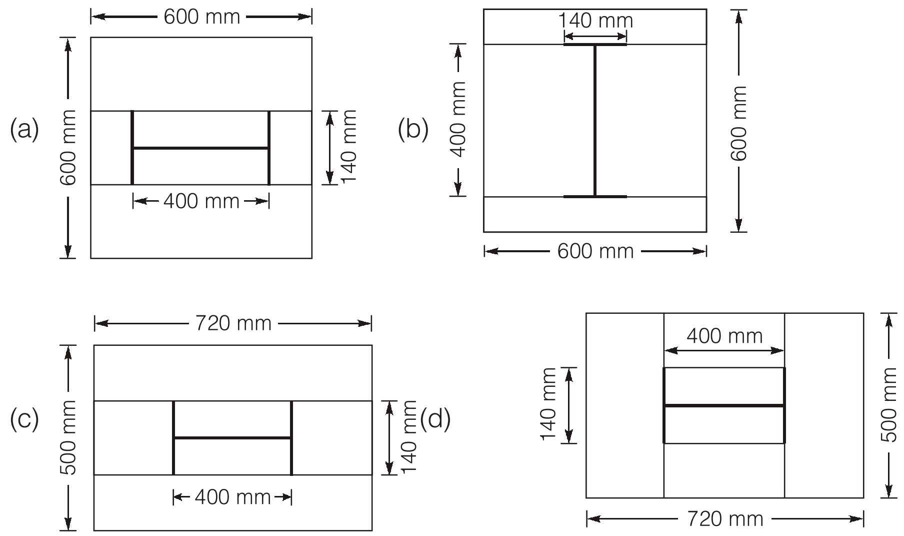
a, b = larger and smaller projection, respectively of the slab base beyond and rectangle circumscribing the column; and

t_f = flange thickness of compression member.

When only the effective area of the base plate is used as in **7.4.1.1**, c^2 may be used in the above equation instead of $(a^2 - 0.3b^2)$.

Example:

Q.1 Which one of the following plan views of a gusseted base plate will result in minimum base plate thickness?



[IES : 2000]

Ans. (b)

Plate thickness in gusseted plate base,

$t = c\sqrt{3w/\sigma_{bs}}$

$c \rightarrow$ portion of the base plate acting as a cantilever

$t \rightarrow$ thickness of base plate at critical section

$w \rightarrow$ intensity of pressure from concrete under the slab

$s_{bs} \rightarrow$ the permissible bending stress in slab base

Figure	Cantilever portion (c)
(a)	$\frac{600-140}{2} = 230$ mm
(b)	100 mm
(c)	180 mm
(d)	160 mm

Thus thickness of base plate shown in figure(b) will be minimum.

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

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

7.5 ANGLE STRUTS

7.5.1 Single Angle Struts

The compression in single angles may be transferred either concentrically to its centroid through end gusset or eccentricity by connecting one of its legs to a gusset or adjacent member.

7.5.1.1 Concentric Loading

When a single angle is concentrically loaded in compression, the design strength may be evaluated using **7.1.2**.

7.5.1.2 Loaded Through One Leg

The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e as given below:

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

where, k_1, k_2, k_3 = constant depending upon the end condition, as given in Table 12

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}} \text{ and } \lambda\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}}$$

where, l = center-to-center length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1, b_2 = width of the two legs of the angle,

t = thickness of the leg, and

ε = yield stress ratio $(250/f_y)^{0.5}$

Table 12 Constants k_1, k_2 and k_3

S.No. (1)	Number of bolts at each end connection (2)	Gusset/connecting member fixity (1) (3)	k_1 (4)	k_2 (5)	k_3 (6)
(i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
(ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

(1) "Stiffness of in-plane rotational restraint provided by the gusset/connecting member.
For partial restraint, the λ_e can be interpolated between the λ_e results for fixed for fixed and hinged cases.

7.5.2 Double Angle Struts

7.5.2.1 For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets 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 end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, KL, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive stress shall not exceed the values based on **7.1.2**.

7.5.2.2 Double angle discontinuous struts connected back-to-back, to one side of a gusset or section by one or more bolts or rivets in each angle, or by the equivalent in welding, shall be designed in accordance with **7.5.1**.

7.5.3 Continuous Members

Double angle continuous struts such as those forming the flanges, chords or ties of trusses or trussed girders, or the legs of towers shall be designed as axially loaded compression members, and the effective length shall be taken in accordance with **7.2.4**.

7.6 LACED COLUMNS

7.6.1 General

7.6.1.1 Members comprising two main components laced and tied, should 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 10 A and 10 B).

Example:

Q.1 Why are tie plates provided in laced columns?

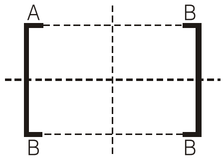
(a) To check the buckling of column as a whole
(b) To check the buckling of the lacing flats
(c) To check the buckling of the component columns
(d) To check the distortion of the end cross sections

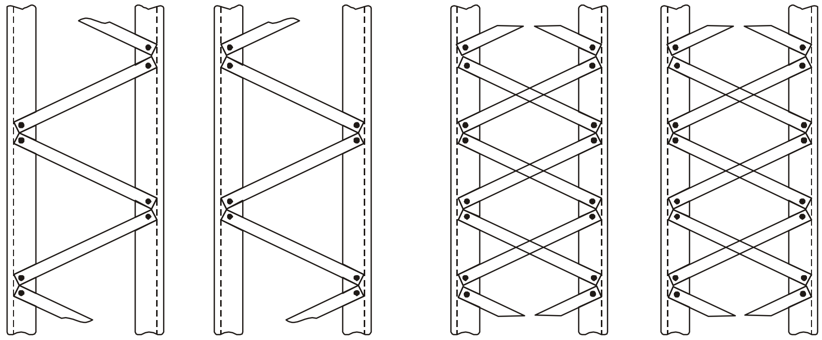
[IES : 2005]

Ans. (d)

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 **7.7**, double laced systems (see figure 10 B) and single laced systems (see figure 10 A) 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 10 C).

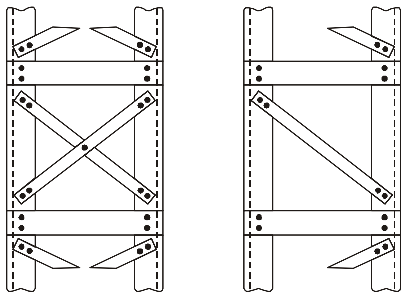




Lacing on face A   Lacing on face B   Lacing on face A   Lacing on face B

Preferred Lacing Arrangement
10A Single laced system

Preferred Lacing Arrangement
10B Double laced system



10C Double laced and single laced system combined with cross numbers

Laced Columns

Example:

Q.1 **Assertion (A) :** In double-laced system of a built-up column, cross member perpendicular to the longitudinal axis of the column is not used.
Reason (R) : Lacing bars are forced to share the axial load on the strut.
[IES : 2003]

Ans. (b)
The purpose of lacing is to hold the various parts of a column straight, parallel, at a correct distance apart and equalize the stress distribution between its various parts. Double flat lacings with battens (cross-members) is not recommended as it gives undesirable effects.

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/riveted construction, the minimum width of lacing bars shall be three times the nominal diameter of the end bolt/rivet.

7.6.3 Thickness of Lacing Bars

The thickness of flat lacing bars shall not be less than one-fortieth of its effective length for single lacing and one-sixtieth of the effective for double lacings.

7.6.4 Angle of Inclination

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.

Example:

Q.1 **Assertion (A) :** Angle of inclination of lacing bars in a built-up column is constrained as 70° > f > 40°, where f is angle of lacing with vertical.

Reason (R) : When this limit is not maintained, the total length of the bar will be large.

(a) both A and R are true and R is the correct explanation of A
(b) both A and R are true but R is not a correct explanation of A
(c) A is true but R is false
(d) A is false but R is true

[IES : 2006]

Ans. (c)

Length will be small when f is greater than 70°. Angle of inclination of lacing bars be such that they do not transmit axial column load from one component to another through secondary truss action. If the lacing is too steep then it is likely to transmit some axial load and if the inclination with axis is large, then its capacity to carry the transverse load is decreased.

7.6.5 Spacing

7.6.5.1 The maximum spacing of lacing bars whether connected by bolting, riveting 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 unfavourable 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.

Example:

Q.1 A compound column had been fabricated with 4 angles of ISA 50 × 50 × 6 placed at corners of a square 300 mm × 300 mm. The radius of gyration of the angle is 10 mm. For the fabricated column, the overall slenderness ratio is 40. What is the maximum distance between lacing bar attachments at the fabricated columns?

(a) 500 mm (b) 400 mm
(c) 300 mm (d) 280 mm

[IES : 2007]

Ans. (d)

7.6.6 Design of Lacings

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 percent of the axial force in the member and shall be divided equally among all transverse lacing systems in parallel planes.

Example:

Q.1 A steel column in a multi-storeyed building carries an axial load of 125 N. It is built up of 2 ISMC 350 channels connected by lacing. The lacing carries a load of

(a) 125 N (b) 12.5 N
(c) 3.125 N (d) Zero

[IES : 2003]

Ans. (c)

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 The slenderness ratio, KL/r, of the lacing bars shall not exceed 145. In bolted/riveted construction, the effective length of lacing bars for the determination of the design strength shall be taken as the length between the inner and 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 lengths 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 as compression/tension members shall be determined by using the appropriate design stresses, f_{cd} subject to the requirements given in **7.6.3** to **7.6.6** and T_d in **6.1**.*

Example:

Q.1 What is the maximum slenderness ratio of lacing bars in built-up columns?

(a) 120 (b) 145
(c) 180 (d) 200

[IES : 2004]

Ans. (b)

7.6.7 Attachment to Main Members

The bolting, riveting 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 of the thickness of the element of the members to which it is connected, whichever is less. The welding be provided along each side of the bar for the full length of lap.

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.

Example:

Q.1 Consider the following statements:

1. As far as practicable, the lacing system shall not be varied throughout the length of the strut.

2. Single laced systems on opposite sides of the components shall preferably be in mutually opposite directions, so that one is not the shadow of the other.
3. Rolled sections or tubes of equivalent strength may be used as lacing bars instead of flats.

Which of these statements are correct?

(a) 1 and 2 (b) 2 and 3
(c) 1 and 3 (d) 1, 2 and 3

[IES : 2004]

Ans. (c)

Q.2 Consider the following stipulations in designing a laced column:

1. Single lacing systems on opposite planes shall preferably be in the same direction so that one is the shadow of the other.
2. Lacing bar should be a flat section.
3. The slenderness ratio of the lacing bars for compression shall not exceed 180.
4. Laced compression members are to be provided with tie plates at ends.

Which of these observations is/are correct?

(a) 1 only (b) 1 and 3
(c) 2 and 4 (d) 1 and 4

[IES : 2010]

Ans. (d)

Q.3 In laced columns, end tie-plates are provided to

(a) check the buckling column
(b) keep the column components in position
(c) check the distortion of column sections at ends because of unbalanced horizontal force from lacings.
(d) prevent rotation of elements

[IES : 2012]

Ans. (c)

7.7 BATTENED COLUMNS

7.7.1 General

7.7.1.1 Compression members composed of two main components battened should preferably have the individual members of the same cross-section and symmetrically disposed about their major axis. Where practicable, the compression members should 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 Fig. 11).

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 bays within its actual length from center-to-center of end connections.

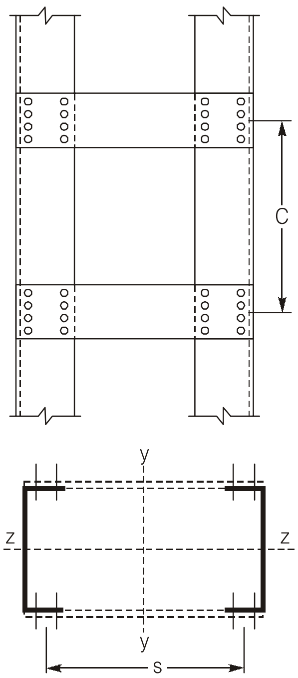


Fig. 11 Batten column section

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 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 riveted, bolted or welded to the main components so as to resist simultaneously a shear $V_b = V_tC/NS$ along the column axis and a moment $M = V_tC/2N$ 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 rivet/bolt group/welding connecting the batten to the main member.

Example:

Q.1 **Assertion (A) :** Battening of columns shall be done where the columns are subjected to eccentric loading in the plane of battens.
Reason (R) : Batten plates are designed to resist moments and longitudinal forces arising due to transverse shear force.

(a) both A and R are true and R is the correct explanation of A
(b) both A and R are true but R is not a correct explanation of A
(c) A is true but R is false
(d) A is false but R is true

[IES : 2000]

Ans. (d)

Q.2 Which one of the following forces is used for the design of battens of a built-up column?

(a) Axial load
(b) Twisting momen
(c) Vertical shear
(d) Transverse shear

[IES : 2008]

Ans. (d)

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 at a tie plate and its fastening 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, rivets 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 or rivets, 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 of flanges perpendicular to the main member. However, it should be ensured that the ends of the compression members are tied to achieve adequate rigidity.

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 on 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.

SECTION 8 : DESIGN OF MEMBERS SUBJECTED TO BENDING

Example:

Q.1 Which of the following statements are correct?

1. In a steel compound column section, the width will be smaller when they are placed face to face than when they are placed back to back.
2. In the design of steel compound columns the length of battens are normally longer than the lacings.
3. Lacings in a steel compound column are designed as slender compression members.

(a) 1 and 3 only (b) 1 and 2 only
(c) 2 and 3 only (d) 1, 2 and 3

[IES : 2013]

Ans. (a)

8.1 GENERAL

Members subjected to predominant bending shall have adequate design strength to resist bending moment, shear force, and concentrated forces imposed upon and their combinations. Further, the members shall satisfy the deflection limitation presented in Section 5, as serviceability criteria.

8.1.1 Effective Span of Beams

The effective span of a beam shall be taken as the distance between the center 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. 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.

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 Lateral 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 is not susceptible to web buckling under shear before yielding (where $d/t_w \le 67\varepsilon$) 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 to semi-compact but the web is susceptible to shear buckling before yielding ($d/t_w \le 67\varepsilon$), 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.

(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 0.6 V_d, where V_d is the design shear strength of the cross-section, the design bending strength, M_d shall be taken as:

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

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

where, β_b = 1.0 for plastic and compact sections

β_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; and

γ_{m0} = partial safety factor

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

$$M_d = M_{dv}$$

where, M_{dv} = design bending strength under high shear as per design of section under combined loading in section 9.

Example:

Q.1 A cantilever steel beam of 3 m span carries a uniformly distributed load of 20 kN/m inclusive of self weight. The beam comprises ISLB 200@198 N/mm flange100 mm × 7.3 mm, web thickness 5.4 mm

I_{xx} = 1696.6 cm^4, I_{yy} = 115.4 cm^4

Bending and shear stresses in the beam are respectively

(a) 530.47 N/mm^2 and 55.55 N/mm^2

(b) 3899.48 N/mm^2 and 82.19 N/mm^2

(c) 132.62 N/mm^2 and 41.1 N/mm^2

(d) 1949.74 N/mm^2 and 41.10 N/mm^2

[IES : 1995]

Ans. (a)

The maximum bending moment,

$$M = \frac{wl^2}{2} = \frac{20 \times 3^2}{2} = 80 \text{ kN-m}$$

Section modulus of beam,

$$Z = \frac{I_{xx}}{(200/2)} = \frac{1696.6 \times 10^4}{100}$$

$$= 1696.6 \times 10^2 \text{ mm}^3$$

$$\text{Bending stress, } \sigma = \frac{M}{Z} = \frac{80 \times 10^6}{1696.6 \times 10^2}$$

$$= 530.47 \text{ N/mm}^2$$

Maximum shear force,

$$V = wl = 20 \times 3 = 60 \text{ kN}$$

$$\text{Shear stress, } \tau = \frac{V}{t_w d}$$

Thickness of web,

$$t_w = 5.4 \text{ mm}$$

$$\therefore \quad \tau = 60 \times 10^3/(5.4 \times 200)$$

$$= 55.5 \text{ N/mm}^2$$

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

$$\left(\frac{A_{nf}}{A_{gf}}\right) \geq \frac{\left(\frac{f_y}{f_u}\right)\left(\frac{\gamma_{m1}}{\gamma_{m0}}\right)}{0.9}$$

where

$\frac{A_{nf}}{A_{gf}}$ = ratio of net to gross area of the flange in tension,

$\frac{f_y}{f_u}$ = ratio of yield and ultimate stress of the material, and

$\frac{\gamma_{m1}}{\gamma_{m0}}$ = ratio of partial safety factors against ultimate to yield stress (see 5.4.1).

When the $\frac{A_{nf}}{A_{gf}}$ 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 to 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), $b_0 \le \frac{L_0}{20}$; and

(b) For internal elements (supported along two edges), $b_1 \le \frac{L_0}{10}$.

where, L_0 = length between points of zero moment (inflection) in the span,
b_0 = width of the flange with outstand, and
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) 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, β_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
f_{bd} = design bending compressive stress, obtained as given below
$f_{bd} = \chi_{LT} f_y/\gamma_{m0}$
χ_{LT} = bending stress reduction factor to account for lateral torsional buckling, given by:

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \le 1.0$$

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

α_{LT}, the imperfection parameter is given by
α_{LT} = 0.21 for rolled steel section
α_{LT} = 0.49 for welded steel section

The non-dimensional slenderness ratio, λ_{LT}, is given by

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \le \sqrt{1.2 Z_e f_y / M_{cr}}$$

$$= \sqrt{\frac{f_y}{f_{cr,b}}}$$

where, M_{LT} = elastic critical moment calculated in accordance with **8.2.2.1**, and

$f_{cr,\,b}$ = extreme fibre bending compressive stress corresponding to elastic lateral 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:

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(L_{LT})^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]\right\}} = \beta_b Z_p f_{cr,b}$$

b of non-slender rolled steel sections in the above equation may be approximately calculated using the following equation:

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

The following simplified equation may be used in the 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}.

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

where, I_t = torsional constant = $\dfrac{\Sigma b_i t_i^3}{3}$ 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 = center-to-center distance between flanges; and

t_f = thickness of the flange.

8.3 EFFECTIVE LENGTH FOR LATERAL TORSIONAL BUCKLING

8.3.1 For simply supported beams and girders of span length L, where no lateral restraint to the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective length L_{LT} of the lateral buckling to be used in **8.2.2.1** shall be taken as in Table 15.

Table 15 Effective Length for Simply Supported Beams, L_{LT}

(Clause 8.3.1)

Sl.No.	Conditions of Restraint at Supports		Loading Condition	
	Torsional Restraint	Warping Restraint	Normal	Destabilizing
(1)	(2)	(3)	(4)	(5)
(i)	Fully restrained	Both flanges fully restrained	0.70 L	0.85 L
(ii)	Fully restrained	Compression flange fully restrained	0.75 L	0.90 L
(iii)	Fully restrained	Both flanges fully restrained	0.80 L	0.95 L
(iv)	Fully restrained	Compression flange partially restrained	0.85 L	1.00 L
(v)	Fully restrained	Warping not restrained in both flanges	1.00 L	1.20 L
(vi)	Partially restrained by bottom flange	Warping not restrained in both flanges	1.0 L + 2 D	1.2 L + 2 D
(vii)	support connection			
(viii)	Partially restrained by bottom flange bearing support	Warping not restrained in both flanges	1.2 L + 2 D	1.4 L + 2 D

Notes:
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.

In simply supported beams with intermediate lateral restraints against lateral torsional buckling, the effective length for lateral torsional buckling to be used in **8.2.2.1**, L_{LT} shall be taken as the length of the relevant segment in between the lateral restraints. The effective length shall be equal to 1.2 times the length of the relevant segment in between the lateral restraints.

Restraint against torsional rotation at supports in these beams may be provided by:

(a) web or flange cleats, or
(b) bearing stiffeners acting in conjunction with the bearing of the beam, or
(c) lateral end frames or external supports providing lateral restraints to the compression flanges at the ends, or
(d) their being built into walls.

Example:

Q.1 In a simply supported beam of span L each end is restrained against torsion, compression flange being unrestrained. According to IS: 800, the effective length of the compression flange will be equal to

(a) L (b) 0.85 L
(c) 0.75 L (d) 0.70 L

[IES : 1999]

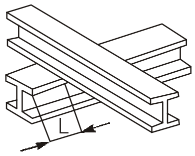
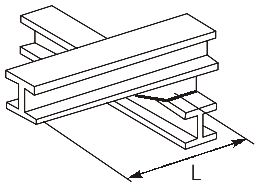
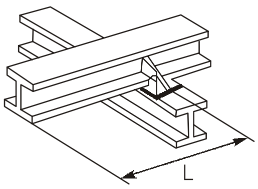
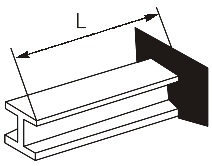
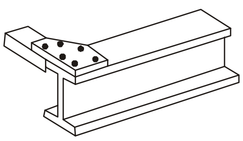
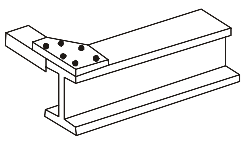
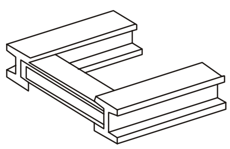
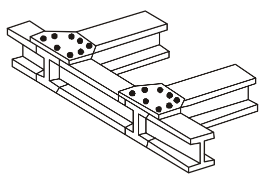
Ans. (a)

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 lengths for lateral torsional buckling shall be taken as the distance, center-to-center 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 center 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 16 for different support conditions.

Table 16 Effective Length, L_{LT} for Cantilever of Length, L

(Clause 8.3.3)

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

8.4 SHEAR

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

$$V \le V_d$$

where, V_d = design strength

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 or strength of the web as governed by shear buckling.

8.4.1 The nominal plastic shear resistance under pure shear is given by

$$V_n = V_p$$

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

A_v = shear area, and

f_{yw} = yield strength of the web

8.4.1.1 The shear area may be calculated as given below:

I and channel sections:

Major Axis Bending

Hot-Rolled – ht_w

Welded – dt_w

Minor Axis Bending

Hot-Rolled or Welded – $2b\,t_f$

Rectangular hollow sections of uniform thickness:

Loaded parallel to depth (h) – A h/(b + h)

Loaded parallel to width (b) – A b/(b + h)

Circular hollow tubes of uniform thickness – 2 A/π

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, and

t_w = thickness of the web

Note: Fastener holes need not be accounted for in plastic design shear strength calculation provided that:

$$A_{vn} \ge (f_y/f_u)\,(\gamma_{m1}/\gamma_{m0})\,A_v/0.9$$

If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Block shear failure criteria may be verified at the end connections. Section 9 may be referred to for design strength under combined high shear and bending.

8.4.2 Resistance to Shear Buckling

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

$\dfrac{d}{t_w} > 67\varepsilon$ for a web without stiffeners, and $> 67\varepsilon\sqrt{\dfrac{K_v}{5.35}}$ for a web with stiffeners where, K_v = shear buckling coefficient and (see 8.4.2.2)

$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

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. The nominal shear strength is given by

$$V_n = V_{cr}$$

where,

V_{cr} = shear force corresponding to web buckling

$= A_v \tau_b$

where,

τ_b = shear stress corresponding to web buckling, determined as follows:

1. when $\lambda_w \le 0.8$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}}$$

2. when $0.8 < \lambda_w < 1.2$

$$\tau_b = [1 - 0.8(\lambda_w - 0.8)]\left(\frac{f_{yw}}{\sqrt{3}}\right)$$

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

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

$t_{cr,e}$ = the elastic critical shear stress of the web

$$= \frac{K_v \pi^2 E}{12(1-\mu^2)[d/t_w]^2}$$

where, μ = Poisson's ratio, and

K_v = 5.35 when transverse stiffeners are provided only at supports

$= 4.0 + 5.35/(c/d)^2$ for $c/d < 1.0$

$= 5.35 + 4.0/(c/d)^2$ for $c/d \ge 1.0$

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

(b) **Tension field method:** The tension field method, based on the post-shear buckling strength, may be used for webs with intermediate transverse stiffeners, in addition to the transverse stiffeners at supports, provided the panels adjacent to the panel under tension field action, or the end posts provide anchorage for the tension fields and if $c/d \ge 1.0$, where c, d are the spacing of transverse stiffeners and depth of the web, respectively.

In the tension field method, the nominal shear resistance, V_n, is given by

$$V_n = V_{tf}$$

where,

$$V_{tf} = [A_v \tau_b + 0.9\, w_{tf}\, t_w f_v \sin\phi] \le V_p$$

where,

τ_b = buckling strength, as obtained from **8.4.2.2** (a)

f_v = yield strength of the tension field obtained from

$$= \left[f_{yw}^2 - 3\tau_b^2 + \Psi^2\right]^{0.5} - \Psi$$

$\Psi = 1.5\, \tau_b \sin 2\phi$

ϕ = inclination of the tension field

$$= \tan^{-1}\left(\frac{d}{c}\right)$$

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 stiffeners in the web

τ_b = shear stress corresponding to buckling of web **8.4.2.2** (a)

s_c, s_t = anchorage lengths of tension field along the compression and tension flange respectively, obtained from:

$$s = \frac{2}{\sin\phi}\left[\frac{M_{fr}}{f_{yw}\, t_w}\right]^{0.5} \le c$$

where, M_{fr} = reduced plastic moment capacity of the respective flange plane (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:

$$M_{fr} = 0.25 b_f\, t_f^2\, f_{yf}\left[1 - \left\{N_f/(b_f\, t_f\, f_{yf}/\gamma_{m0})\right\}^2\right]$$

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

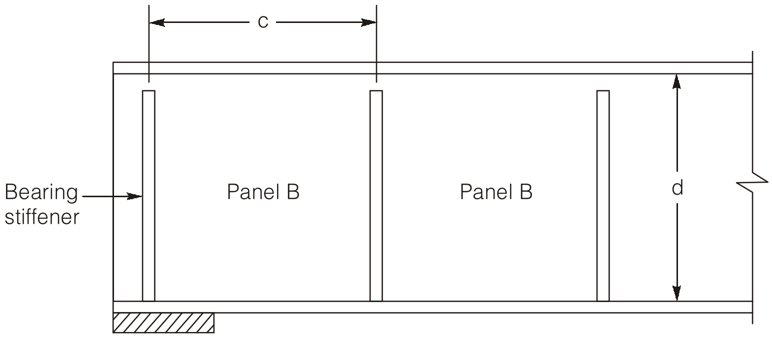
f_{yf} = yield stress of the flange

8.5 Stiffened Web Panels

8.5.1 End Panels Design

The design of end panels in girders in which the interior panel (panel A) is designed using tension field action shall be carried in accordance with the provisions given herein. In this case the end panel should be designed using only simple post critical method, according to **8.4.2.2** (a).

Additionally, the end panel along with the stiffeners should be checked as a beam spanning between the flanges to resist a shear force, R_{tf} and a moment, M_{tf} due to tension field forces as given in **8.5.3**. Further, end stiffener should be capable of resisting the reaction plus a compressive force due to the moment, equal to M_{tf}.



Notes:
1. *Panel A is designed utilizing tension field action as given in **8.4.2.2** (b).*
2. *Panel B is designed without utilizing tension field action as given in **8.4.2.2** (a).*
3. *Bearing stiffener is designed for the compressive force due to bearing plus compressive force due to the moment M_{tf} as given in **8.5.3**.*

Fig. 12 End panel designed not using tension field action

8.5.2 End Panels Designed Using Tension Field Action

The design of end panels in girders, which are designed using tension field action shall be carried out in accordance with the provisions mentioned herein. In this case, the end panel (Panel B) shall be designed according to **8.4.2.2** (b).

Additionally it should be provided with an end post consisting of a single or double stiffener, satisfying the following:

(a) **Single stiffener:** The top of the end post should be rigidly connected to the flange using full strength welds. (See Fig. 13)
The end post should be capable of resisting the reaction plus a moment from the anchor forces equal to 2/3 M_{tf} due to tension field forces, where M_{tf} is obtained from **8.5.3**. The width and thickness of the end post are not to exceed the width and thickness of the flange.

(b) **Double stiffener:** The end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force R_{tf} and a moment, M_{tf} due to the tension field forces as given in **8.5.3**. (See Fig. 14)

8.5.3 Anchor Forces

The resultant longitudinal shear, R_{tf}, and a moment M_{tf} from the anchor of tension field forces are evaluated as given below:

$$R_{tf} = \frac{H_q}{2} \text{ and } M_{tf} = \frac{H_q d}{10}$$

where,
$$H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$$

$$V_p = \frac{d t f_y}{\sqrt{3}}$$

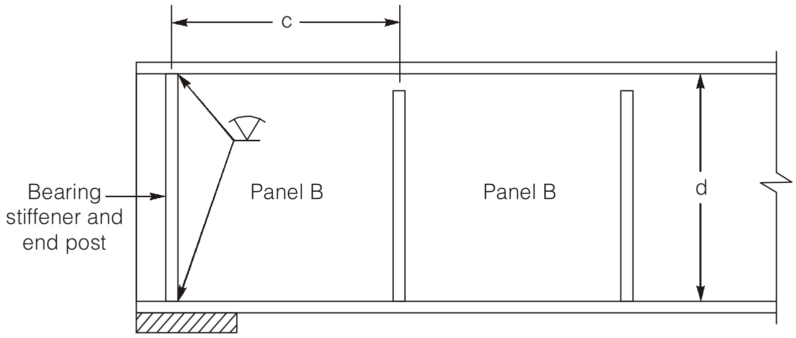
d = web depth

If the actual factored shear force, V in the panel designed using tension field approach is less than the shear strength, V_{tf} as given in **8.4.2.2** (b), 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 as given in **8.4.2.2** (b), and

V_{cr} = critical shear strength for the panel designed utilizing tension field action as given in **8.4.2.2** (a).

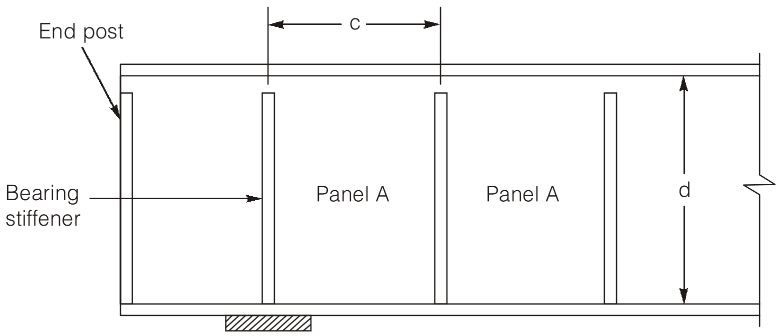
8.5.4 **Panels with Opening:** Panels with opening of dimension greater than 10 percent of the minimum panel dimension should be designed without using tension field action as given in **8.4.2.2** (b). The adjacent panels should be designed as an end panel as given in **8.5.1** or **8.5.2**, as appropriate.



Notes:

1. *Panel A is designed utilizing tension field action as given in **8.4.2.2** (b).*
2. *Panel B is designed utilizing tension field action as given in **8.4.2.2** (b).*
3. *Bearing stiffener and end post is designed for combination of compressive loads due to bearing and a moment equal to 2/3 M_{tf} as given in **8.5.3**.*

Fig. 13 End panel designed using tension field action (Single stiffener)



Notes:

1. *Panel A is designed utilizing tension field action as given in **8.4.2.2** (b).*
2. *Bearing stiffener is designed for compressive force due to bearing as given in **8.4.2.2** (a).*
3. *End post is designed for horizontal shear R_{tf} and moment M_{tf} as given in **8.5.3**.*

Fig. 14 End panel designed using tension field action (Double stiffener)

Example:

Q.1 The allowable shear stress in stiffened webs of mild steel beams decreases with
(a) decrease in the spacing of the stiffeners
(b) increase in the spacing of the stiffeners
(c) decrease in the effective depth
(d) increase in the effective depth

[IES : 1997]

Ans. (b)

8.6 DESIGN OF BEAMS AND PLATE GIRDERS WITH SOLID WEBS

8.6.1 Minimum Web Thickness

The thickness of the web in a section shall satisfy the following requirements:

8.6.1.1 Serviceability Requirement

(a) When transverse stiffeners are not provided, $\frac{d}{t_w} \le 200\,\varepsilon$ (web connected to flanges along both longitudinal edges).

$\frac{d}{t_w} \le 90\,\varepsilon$ (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),

1. when $3d \ge c \ge d$

$$\frac{d}{t_w} \le 200\,\varepsilon$$

2. when $0.74\,d \le c < d$

$$\frac{c}{t_w} \le 200\,\varepsilon_w$$

3. when $c < d$

$$\frac{d}{t_w} \le 270\,\varepsilon_w$$

4. when $c > 3d$, the web shall be considered as unstiffened

(c) When transverse stiffeners and longitudinal stiffeners at one level only are provided (0.2 d from compression flange) according to **8.7.13** (a).

1. when $2.4\,d \ge c \ge d$

$$\frac{d}{t_w} \le 250\,\varepsilon_w$$

2. when $0.74\,d \le c \le d$

$$\frac{c}{t_w} \le 250\,\varepsilon_w$$

3. when $c < 0.74\,d$

$$\frac{d}{t_w} \le 340\,\varepsilon_w$$

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

$$\frac{d}{t_w} \le 4000\,\varepsilon_w$$

where, d = depth of the web,
t_w = thickness of the web,
c = spacing of transverse stiffener
ε_w = yield stress ratio of web = $\sqrt{\frac{250}{f_{yw}}}$ and
f_{yw} = yield stress of the web

Example:

Q.1 The thickness of web for unstiffened plate girder with clear distance 'd' between the flanges shall not be less than

(a) d/200 (b) d/85
(c) d/100 (d) d/160

[IES : 1996]

Ans. (b)

Q.2 In a plate girder bridge the thickness of web is less than d′/200 where d′ is the unsupported depth of web. The web plate should be provided with

(a) vertical stiffeners
(b) horizontal stiffeners
(c) end stiffeners
(d) both vertical and horizontal stiffeners

[IES : 2001]

Ans. (d)

8.6.1.2 Compression Flange Buckling Requirement

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

(a) When transverse stiffeners are not provided

$$\frac{d}{t_w} \le 345\,\varepsilon_f^2$$

(b) When transverse stiffeners are provided and

1. when $c \ge 1.5\,d$

$$\frac{d}{t_w} \le 345\,\varepsilon_f^2$$

2. when $c < 1.5\,d$

$$\frac{d}{t_w} \le 345\,\varepsilon_f$$

where, d = depth of the web
t_w = thickness of the web
c = spacing of transverse stiffener
ε_f = yield stress ratio of web = $\sqrt{\frac{250}{f_{yf}}}$, and
f_{yf} = yield stress of compression flange

8.6.2 Sectional Properties

8.6.2.1 The effective sectional area of compression flanges shall be the gross area with deductions for excessive width of plates as specified for compression members and for open holes occurring in a plane perpendicular to the direction of stress at the section being considered.
The effective sectional area of tension flanges shall be the gross sectional area with deductions for holes as specified in **8.2.1.4**.
The effective sectional area for parts in shear shall be taken as specified in **8.4.1.1**.

8.6.3 Flanges

8.6.3.1 In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not less than one-third) and the number of flange plates shall be kept to a minimum.
In exposed situations, where flange angles are used, at least one plate of the top flange shall extend over the full length of the girder.
Each flange plate shall extend beyond its theoretical cut-off point, and the extension shall contain sufficient rivets, bolts or welds to develop in the plate, the load calculated for the bending moment on the girder section (taken to include the curtailed plate) at the theoretical cut-off point.

8.6.3.2 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 center of gravity shall coincide, as nearly as possible, with that of the element spliced. There shall be enough bolts, rivets or welds on each side of the splice to develop to 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 but welds, wherever possible. These butt welds shall developed the full strength of the plates.

8.6.3.3 Connection of Flanges to Web

The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the maximum horizontal shear force resulting from the bending moment gradient in the girder, combined with any vertical loads which are directly applied to the flange. If the web is designed using tension field methods as given in **8.4.2.2** (b), the weld should be able to transfer the tension field stress, f_{yw} acting on the web.

8.6.3.4 Bolted/Riveted Construction

For girders in exposed situations and which do not have flange plates for their entire length, the top edge, of the web plate shall be flush with or above the angles, and the bottom edge of the web plate shall be flush with or set back from the angles.

8.6.3.5 Welded Construction

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.

8.6.4.2 Splices in Webs

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

Splices in the webs of the plate girders and rolled sections shall be designed to resist the shears and moments all the spliced section.

Example:

Q.1 At a section along the span of a welded plate girder, where the web is spliced, the bending moment is M. If the girder has top flange, web and bottom flange plates of equal area, then the share of the bending moment which would be taken by the splice plates would be

(a) M (b) M/3
(c) M/7 (d) M/13

[IES : 1997]

Ans. (a)

8.7 STIFFENER DESIGN

8.7.1 General

8.7.1.1 When the web of a member acting alone (that is without stiffeners) proves inadequate, stiffeners for meeting the following requirements should be provided:

(a) **Intermediate transverse web stiffener:** To improve the buckling strength of a slender web due to shear.

(b) **Load carrying stiffener:** To prevent local buckling of the web due to concentrated loading.

(c) **Bearing stiffener:** To prevent local crushing of the web due to concentrated loading.

(d) **Torsion stiffener:** To provide torsional restraint to beams and girders at supports.

(e) **Diagonal stiffener:** To provide local reinforcement to a web under shear and bearing.

(f) **Tension stiffener:** To transmit tensile forces applied to web through a flange.

The same stiffeners may perform more than one function and their design should comply with the requirements of all the functions for which designed.

8.7.1.2 Outstand of Web Stiffeners

Unless the outer edge is continuously stiffened, the outstand from the face of the web should not exceed $20\, t_q\, \varepsilon$.

When the outstand of web is between $14\, t_q \varepsilon$ and $20\, t_q \varepsilon$, then the stiffener design should be on the basis of a core section with an outstand of $14\, t_q \varepsilon$, where t_q is the thickness of the stiffener.

8.7.1.3 Stiff Bearing Length

The stiff bearing length of any element b_1, is that length which cannot deform appreciably in bending. To determine b_1, 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 Fig. 15)

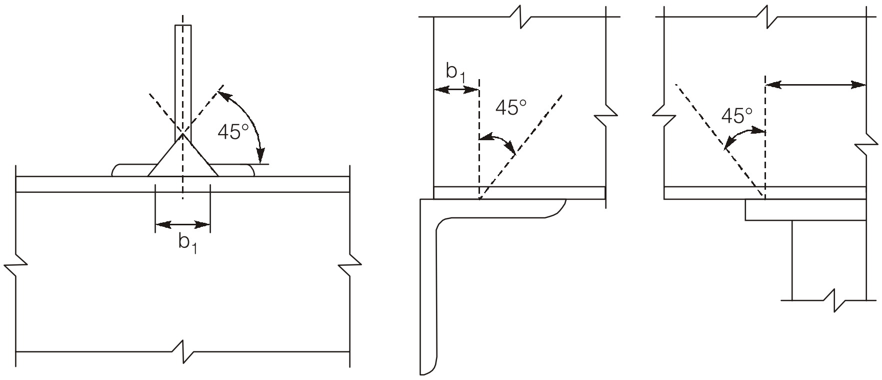


Fig. 15 Stiff bearing length, b_1

8.7.2 Design of Intermediate Transverse Web Stiffeners

8.7.2.1 General

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

8.7.2.2 Spacing

Spacing of intermediate stiffeners, where provided, shall comply with 8.6.1 depending on the thickness of the web.

8.7.2.3 Outstand of Stiffeners

The outstand of the stiffeners should comply with **8.7.1.2**.

8.7.2.4 Minimum Stiffeners

Transverse web stiffeners not subject to external loads or moments should have a second moment of area, I_s about the centreline of the web, if stiffeners are on both sides of the web and about the face of the web, if single stiffener on only one side of the web is used such that:

if $\dfrac{c}{d} \geq \sqrt{2}$, $\quad I_s \geq 0.75\, d t_w^3$, and

if $\dfrac{c}{d} < \sqrt{2}$, $\quad I_s \geq \dfrac{1.5\, d^3\, t_w^3}{c^2}$

where, d = depth of the web;

t_w = minimum required web thickness for spacing using tension field, as given in **8.4.2.1**; and

c = actual stiffener spacing

8.7.2.5 Buckling check on intermediate transverse web stiffeners.

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

$$F_q = \frac{V - V_{cr}}{\gamma_{m0}} \le F_{qd}$$

where, F_{qd} = design resistance of the intermediate stiffeners
V = factored shear force adjacent to the stiffener, and
V_{cr} = shear buckling resistance of the web panel designed without using tension field action as given in **8.4.2.2** (a).

Stiffeners subject to external loads and moments should meet the conditions for load carrying web stiffeners in **8.7.3**. In addition they should satisfy the following interaction expression:

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

If $F_q < F_x$, then $(F_q - F_x)$ should be taken as zero;

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 axis parallel to the web.
M_q = moment on the stiffener due to eccentricity 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.

8.7.2.6 Connection of Intermediate Stiffeners to Web

Intermediate transverse stiffeners not subject to external loading should be connected to the web so as to withstand a shear between each component of the stiffener and the web (in kN/mm) of not less than:

$$\frac{t_w^2}{(5b_s)}$$

where, t_w = web thickness, in mm; and
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 $4t_w$.

8.7.4 Bearing Stiffeners

Bearing stiffeners should be provided for webs where forces 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

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

where, b_1 = stiff bearing length

n_2 = length obtained by dispersion through the flange to the web junction at a slope of 1 : 2.5 to the plane of the flange

t_w = thickness of the web, and

f_{yw} = yield stress of the web

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

8.7.8 Design of Tension Stiffeners

Tension stiffeners should be designed to carry the portion of the applied load or reaction less than 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) Conditions of **8.7.4**, and

(b) Second moment of area of the stiffener section about the centerline of the web, I_s should be such that:

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

where,

α_s = 0.006 for $L_{LT}/r_y \leq 50$

= $0.3/(L_{LT}/r_y)$ for $50 < L_{LT}/r_y = 100$

= $30/(L_{LT}/r_y)^2$ for $L_{LT}/r_y > 100$

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, and

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

8.7.13 Horizontal Stiffeners

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

(a) One horizontal stiffener shall 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 when the thickness of the web is less than the limits specified in **8.6.1**. The stiffener shall be designed so that I_s is not less than $4ct_w^3$ where I_s and t_w are as defined in **8.7.2.4** and c is the actual distance between the vertical stiffeners.

(b) A second horizontal stiffener (single or double) shall be placed at the neutral axis of the girder when the thickness of the web is less than the limit specified in **8.6.1**. This stiffener shall be designed so that I_s is not less than $d_2\, t_w^3$ where I_s and t_w are as defined in **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 need not be continuous over them; and

(d) Horizontal stiffeners may be in pairs arranged on each side of the web, or single located on one side of the web.

SECTION 10 : CONNECTIONS

10.1 GENERAL

10.1.1 This section deals with the design and detailing requirements for joints between members. Connection elements consist of component such as cleats, gusset plates, brackets, connecting plates and connectors such as rivets, bolts, pins, and welds. Connections shall be capable of transmitting the calculated design actions.

10.1.2 Where members are connected to the surface of a web or the flange of a section, the ability of the web or the flange to transfer the applied forces locally should be checked and where necessary, local stiffening provided.

10.1.3 Ease of fabrication and erection should be considered in the design of connections. Attention should be paid to clearances necessary for field erection, tolerances, tightening of fasteners, welding procedures, subsequent inspection, surface treatment and maintenance.

10.1.5 In general, 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 fasteners 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 necessary pretension after welding.

10.2 LOCATION DETAILS OF FASTENERS

10.2.1 Clearances for Holes for Fasteners

Bolts may be located in standard size, over size, short slotted or long slotted hole.

(a) **Standard clearance hole:** Except where fitted bolts, bolts in low-clearance or oversize holes are specified, the diameter of standard clearance holes for fasteners shall be as given in Table 19.

(b) **Over size hole:** Holes of size larger than the standard clearance holes, as given in Table 19 may be used in slip resistance connections and hold down bolted connections, only where specified, provided the over size holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole not larger than the standard clearance hole (and hardened washer in slip resistant connections).

(c) **Short and long slots:** Slotted holes of size larger than the standard clearance hole, as given in Table 19 may be used in slip resistant connections and hold down bolted connections, only where specified, provided the over size holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole of size not larger than the standard clearance hole (and hardened washer in slip resistant connection).

Table 19 Clearance for Fastener Holes
(Clause 10.2.1)

Sl.No.	Nominal size of Fastener, d mm	Size of the Hole = Nominal Diameter of the Fastener + Clearance mm			
		Standard clearance in diameter and width of slot	Over size clearance in diameter	Clearance in the length of the slot	
				Short slot	Long slot
(1)	(2)	(3)	(4)	(5)	(6)
(i)	12 – 14	1.0	3.0	4.0	2.5 d
(ii)	16 – 22	2.0	4.0	6.0	2.5 d
(iii)	24	2.0	6.0	8.0	2.5 d
(iv)	Larger than 24	3.0	8.0	10.0	2.5 d

10.2.2 Minimum Spacing

The distance between center of fasteners shall not be less than 2.5 times the nominal diameter of the fastener.

10.2.3 Maximum Spacing

10.2.3.1 The distance between the centers of any two adjacent fasteners shall not exceed 32t or 300 mm, whichever is less, where t is the thickness of the thinner plate.

10.2.3.2 The distance between the centers of two adjacent fasteners (pitch) in the line lying in the direction of stress, shall not exceed 16t or 200 mm, whichever is less, in tension members and 12t or 200 mm, whichever is less, in compression members; where t is the thickness of the thinner plate. In the case of compression members wherein forces are transferred through butting faces, this distance shall not exceed 4.5 times the diameter of the fasteners for a distance equal to 1.5 times the width of the member from the butting faces.

Example:

Q.1 The centre to centre maximum distance between bolts in tension member of thickness 10 mm is

(a) 200 mm (b) 160 mm
(c) 120 mm (d) 100 mm

[IES : 2003]

Ans. (b)

10.2.3.3 The distance between the centers of any two consecutive fasteners in a line adjacent and parallel to an edge of an outside plate shall not exceed 100 mm plus 4t or 200 mm, whichever is less, in compression and tension members; where t is the thickness of the thinner outside plate.

10.2.3.4 When fasteners are staggered at equal intervals and the gauge does not exceed 75 mm, the spacing specified in **10.2.3.2** and **10.2.3.3** between centers of fasteners may be increased by 50 percent, subject to the maximum spacing specified in **10.2.3.1**.

Example:

Q.1 What is the maximum permissible longitudinal pitch in staggered riveted compression joints?

(a) 500 mm (b) 400 mm
(c) 300 mm (d) 100 mm

[IES : 2007]

Ans. (c)

10.2.4 Edge and End Distance

10.2.4.1 The edge distance is the distance at right angles to the direction of stress from the center of a hole to the adjacent edge. The end distance is the distance in the direction of stress from the center of a hole to the end of the element.

In slotted holes, the edge and end distance should be measured from the edge or end of the material to the center of its end radius or the center line of the slot, whichever is smaller. In oversize holes, the edge and end distances should be taken as the distance from the relevant edge/end plus half the diameter of the standard clearance hole corresponding to the fastener, less the nominal diameter of the oversize hole.

10.2.4.2 The minimum edge and end distances from the center of any hole to the nearest edge of a plate shall not be less than 1.7 times the hole diameter in case of sheared or hand-flame cut edges; and 1.5 times the hole diameter in case of rolled, machine-flame cut, sawn and planed edges.

10.2.4.3 The maximum edge distance to the nearest line of fasteners from an edge of any un-stiffened part should not exceed 12 $t\varepsilon$, where $\varepsilon = (250/fy)^{1/2}$ and t is the thickness of the thickness of the thinner outer plate. This would not apply to fasteners interconnecting the components of back to back tension members. Where the members are exposed to corrosive influences, the maximum edge distance shall not exceed 40 mm plus 4t, where t is the thickness of thinner connected plate.

10.2.5 Tacking Fasteners

10.2.5.1 In case of members covered under **10.2.4.3**, when the maximum distance between centers of two adjacent fasteners as specified in **10.2.4.3** is exceeded, tacking fasteners not subjected to calculated stress shall be used.

10.2.5.2 Tacking fasteners shall have spacing in a line not exceeding 32 times the thickness of the thinner outside plate or 300 mm, whichever is less. Where the plates are exposed to the weather, the spacing in line shall not exceed 16 times the thickness of the thinner outside plate or 200 mm, whichever is less. In both cases, the distance between the lines of fasteners shall not be greater than the respective pitches.

10.2.5.4 In tension members composed of two flats, angles, channels or tees in contact back to back or separated back to back by a distance not exceeding the aggregate thickness of the connected parts, tacking fasteners with solid distance pieces shall be provided at a spacing in line not exceeding 1000 mm.

10.2.5.5 For compression members covered in Section 7, tacking fasteners in a line shall be spaced at a distance not exceeding 600 mm.

10.2.6 Countersunk Heads

For countersunk heads, one-half of the depth of the countersinking shall be neglected in calculating the length of the fastener in bearing in accordance with **10.3.3**. For fasteners in tension having countersunk heads, the tensile strength shall be reduced by 33.3 percent. No reduction is required to be made in shear strength calculations.

10.3 BEARING TYPE BOLTS

10.3.1 Effective Areas of Bolts

10.3.1.1 Since threads can occur in the shear plane, the area A_c for resisting shear should normally be taken as the net tensile stress area, A_n of the bolts. For bolts where the net tensile stress area is not defined, A_n shall be taken as the area at the root of the threads.

10.3.1.2 Where it can be shown that the threads do not occur in the shear plane A_c may be taken as the cross-section area, A_s at the shank.

10.3.2 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 10.3.3) and bearing, V_{dpb} (See 10.3.4).

10.3.3 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,
f_u = ultimate tensile strength of a bolt
n_n = number of shear planes with threads intercepting the shear plane
n_s = number of shear planes without threads intercepting the shear plane
A_{sb} = nominal plain shank area of the bolt; and
A_{ab} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread.

10.3.3.2 Large Grip Lengths

When the grip length, l_g (equal to the 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

$$\beta_{lg} = \frac{8d}{(3d + l_g)} = \frac{8}{(3 + l_g/d)}$$

The grip length, lg shall in no case be greater than 8d.

10.3.3.3 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 packing, in mm

10.3.4 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

$= 2.5\, k_b\, d\, t\, f_u$

where,

k_b is smaller of $\dfrac{e}{3d_0}, \dfrac{p}{3d_0} - 0.25, \dfrac{f_{ub}}{f_u}, 1.0;$

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, respectively;

d = nominal diameter of the bolt; and

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 the 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

Notes: The block shear of the edge distance due to bearing force may be checked as given in **6.4**.

10.3.5 Tension Capacity

A bolt subjected to a factored tensile force, T_b shall satisfy:

$$T_b \leq T_{db}$$

where, $T_{db} = \dfrac{T_{nb}}{\gamma_{mb}}$

T_{nb} = nominal tensile capacity of the bolt, calculated as

$0.90\, f_{ub}\, A_u < f_{yb}\, A_{sb}\, (\gamma_{mb}/\gamma_{m0})$

where,

f_{ub} = ultimate tensile stress of the bolt

f_{yb} = yield stress of the bolt

A_n = net tensile stress area as specified in the appropriate Indian Standard (for bolts where the tensile stress area is not defined, A_n shall be taken as the area at the bottom of the threads), and

A_{sb} = shank area of the bolt

10.3.6 Bolt 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

T_b = factored tensile force acting on the bolt, and

T_{db} = design tension capacity

10.4 FRICTION GRIP TYPE BOLTING

10.4.1 In friction grip type bolting, initial pretension in bolt (usually high strength) develops clamping force at the interfaces of elements being jointed. The frictional resistance of elements being jointed. The frictional resistance to slip between the plate surfaces subjected to clamping force oppose slip due to externally applied shear.

10.4.3 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:

$$V_{sf} \le V_{dsf}$$

where,

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}}$$

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows:

$$V_{nsf} = \mu_f\, n_e\, K_h\, F_0$$

where,

μ_f = coefficient of friction (slip factor) as specified in Table 20 ($\mu_f = 0.55$)

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

γ_{mf} = 1.10 (if slip resistance is designed at service load)

= 1.25 (if slip resistance is designed at ultimate load)

F_0 = 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, and

f_0 = proof stress ($= 0.70\, f_{ub}$)

Note: V_{ns} 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.

Table 20 Typical Average Values for Coefficient of Friction (μ_f)

(Clause 10.4.3)

S.No.	Treatment of Surface (2)	Coefficient of Friction, μ_f (3)
(i)	Surface not treated	0.20
(ii)	Surfaces blasted with short or grit with any loose rust removed, no pitting	0.50
(iii)	Surfaces blasted with shot or grit hot-dip galvanized	0.10
(iv)	Surfaces blastd with shot or grit and spray-metallized with zinc (thickness 50-70 μm)	0.25
(v)	Surfaces blasted with short or grit and painted with ethylzinc 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 enthylizinc silicate coat (thickness 60-80 μm)	0.30
(viii)	Surface blasted with shot or grit and painted with alcalizinc silicate coat (thickness 60-80 μm)	0.30
(ix)	Surface based with shot or grit and spray metallized with aluminium (thickness > 50 μm)	0.50
(x)	Clean mill scale	0.33
(xi)	Sand blasted surface	0.48
(xii)	Red lead painted surface	0.1

10.4.5 Tension Resistance

A friction bolt subjected to a factored tension force (T_f) shall satisfy

$$T_f \le T_{df}$$

where, $T_{df} = \dfrac{T_{nf}}{\gamma_{mf}}$

T_{nf} = nominal tensile strength of the friction bolt, calculated as $0.9\, f_{ub} A_n \le f_{yb} A_{sb} (\gamma_{mt} / \gamma_m)$

where,

f_{ub} = ultimate tensile stress of the bolt

A_n = net tensile stress area (for bolts where the tensile stress area is not defined, A_n shall be taken as the area at the root of the threads)

A_{sb} = shank area of the bolt; and

γ_{mf} = partial factor of safety

10.4.6 Combined Shear and Tension

Bolts in a connection for which slip in the serviceability limit state shall be limited, which are subjected to a tension force, T, and shear force, V, shall satisfy

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \le 1.0$$

where, V_{sf} = applied factored shear at design load
V_{df} = design shear strength
T_f = externally applied factored tension at design load, and
T_{df} = design tension strength

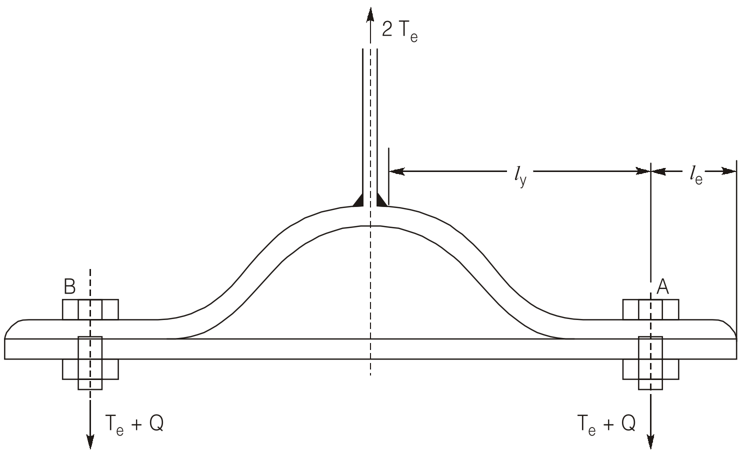
10.4.7 Where prying force, Q as illustrated in figure is significant, it shall be calculated as given below and added to the tension in the bolt

$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_0 b_e t^4}{27 l_e l_v^2}\right]$$

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 of the value given by

$$l_e = 1.1t\sqrt{\frac{\beta f_0}{f_y}}$$

where, β = 2 for non pre-tensioned bolt and 1 for pre-tensioned bolt
η = 1.5
b_e = effective with of flange per pair of bolts
f_0 = proof stress in consistent units, and
t = thickness of the end plate



Combined prying force and tension

10.5 WELDS AND WELDING

10.5.1.1 End Returns

Fillet welds terminating at the ends of sides of parts should be returned continuously around the corners for a distance of not less than twice the size of the weld, unless it is impractical to do so. This is particularly important on the tension end of parts carrying bending loads.

10.5.1.2 Lap Joint

In the case of lap joints, the minimum lap should not be less than four times the thickness of the thinner part joined or 40 mm, whichever is more. Single end fillet should be used only when lapped parts are restrained from openings. When end of an element is connected only by parallel longitudinal fillet welds, the length of the weld along either edge should not be less than the transverse spacing between longitudinal welds.

10.5.1.3 A single fillet weld should not be subjected to moment about the longitudinal axis of the weld.

10.5.2 Size of Weld

10.5.2.1 The size of normal fillets shall be taken as the minimum weld leg size. 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.

10.5.2.2 For fillet welds made by semi-automatic or automatic processes, where the depth of penetration is considerably in excess of 2.4 mm, the size shall be taken considering actual depth of penetration subject to agreement between the purchaser and the contractor.

10.5.2.3 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 21, to avoid the risk of cracking in the absence of preheating.

10.5.2.4 The size of butt weld shall be specified by the effective throat thickness.

10.5.3 Effective Throat Thickness

10.5.3.1 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 special circumstances, where t is the thickness of the thinner plate of elements being welded.

Table 21 Minimum Size of First Run or of a Single Run Fillet Weld

(Clause 10.5.2.3)

Sl.No.	Thickness of Thicker Part (mm)		Minimum Size (mm)
	Over	Up to and including	
(1)	(2)	(3)	(4)
(i)	–	10	3
(ii)	10	20	5
(iii)	20	32	6
(iv)	32	50	8 of first run 10 for minimum size of web

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 prevent cracking of the weld.
2. Where the thicker part is more than 50 mm thick, special precautions like preheating should be taken.

10.5.3.2 For the purpose of stress calculation in fillet welds joining faces inclined to each other, the effective throat thickness shall be taken as K times the fillet size, where K is a constant, depending upon the angle between fusion faces, as given in Table 22.

Table 22 Values of K for Different Angles Between Fusion Faces

(Clause 10.5.3.2)

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

10.5.3.3 The effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined, and that of an incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcements.

10.5.4 Effective Length or Area of Weld

10.5.4.1 The effective length of fillet weld shall be taken as only that length which is of the specified size and required throat thickness. In practice the actual length of weld is made of the effective length shown in drawing plus two times the weld size, but not less than four times the size of the weld.

10.5.4.2 The effective length of butt weld shall be taken as the length of the continuous full size weld, but not less than four times the size of the weld.

10.5.4.3 The effective area of a plug weld shall be considered as the nominal area of the hole in the plane of the faying surface. These welds shall not be designed to carry stresses.

10.5.4.4 If the maximum length *l*j of the side welds transferring shear along its length exceeds 150 times the throat size of the weld, t_t, the reduction in weld strength as per the long joint should be considered. For flange to web connection, where the welds are loaded for the full length, the above limitation would not apply.

10.5.5 Intermittent Welds

10.5.5.1 Unless otherwise specified, the intermittent fillet welding shall have an effective length of not less than four times the weld size, with a minimum of 40 mm.

10.5.5.2 The clear spacing between the effective lengths of intermittent 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.

10.5.5.3 Unless otherwise specified, the intermittent butt weld shall have an effective length of not less than four times the weld size and the longitudinal space between the effective length of welds shall not be more than 16 times the thickness of the thinner part joined. The intermittent welds shall not be used in positions subject to dynamic, repetitive and alternating stresses.

10.5.6 Weld Types and Quality

For the purpose of this code, weld shall be fillet, butt, slot of plug or compound welds.

10.5.7 Design Stresses in Welds

10.5.7.1 Shop Welds

10.5.7.1.1 Fillet Welds

Design strength of a fillet weld, f_{wd} shall be based on its throat area and shall be given by

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

where, $f_{wn} = \frac{f_u}{\sqrt{3}}$

f_u = smaller of the ultimate stress of the weld or of the parent metal, and

γ_{mw} = partial safety factor (see Table 5)

10.5.7.1.2 Butt Welds

Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal.

10.5.7.1.3 Slot or Plug Welds

The design shear stress on slot or plug welds shall be as per **10.5.7.1**.

10.5.7.2 Site Welds

The design strength in shear and tension for site welds made during erection of structural members shall be calculated according to **10.5.7.1** but using a partial safety factor γ_{mw} of 1.5.

10.5.7.3 Long Joints

When the length of the welded joint, l_j of a splice or end connection in a compression or tension element is greater than 150 t_t, the design capacity of weld, f_{wd} shall be reduced by the factor

$$\beta_{jw} = 1.2 - \frac{0.2 l_j}{150 t_t} \leq 1.0$$

where, l_j = length of the joint in the direction of the force transfer, and

t_t = throat size of the weld

10.5.8 Fillet weld applied to the edge of a plate or section.

10.5.8.1 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 arris. (See Fig. 17A)

10.5.8.2 Where the fillet weld is applied to the rounded toe of a rolled section, the specified size of the weld should generally not exceed 3/4 of the thickness of the section at the toe. (See Fig. 17B)

10.5.8.3 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 Fig. 18)

10.5.8.4 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 of section, with the limitations specified in **10.5.8.3**.

10.5.8.5 End fillet weld, 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 Fig. 19. The difference in thickness of the welds shall be negotiated at a uniform slope.

10.5.9 Stresses Due to Individual Forces

When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by

$$f_a \text{ or } q = \frac{P}{t_t\, l_w}$$

where,
f_a = calculated normal stress due to axial force, in N/mm^2
q = shear stress, in N/mm^2
P = force transmitted (axial force N or the shear force Q)
t_t = effective throat thickness of weld, in mm and
l_w = effective length of weld, in mm

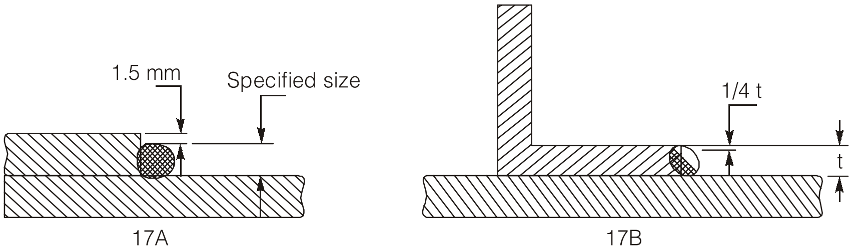


Fig. 17 Fillet welds on square edges of plate or round toe of rolled section

Fig. 18 Full size fillet weld applied to the edge of a plate of section

10.5.10 Combination of Stresses

10.5.10.1 Fillet Welds

10.5.10.1.1 When subjected to a combination of normal and shear stress, the equivalent stress f_e shall satisfy the following:

$$f_e = \sqrt{f_a^2 + 3q^2} \le \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

where,
f_a = normal stresses, compression or tension, due to axial force or bending moment (See 10.5.9), and
q = shear stress due to shear force or tension (see 10.5.9)

10.5.10.1.2 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 10.5.7.1.1)

10.5.10.2. Butt Welds

10.5.10.2.1 Check for the combination of stresses in butt welds need not be carried out provided that:

(a) butt welds are axially loaded, and

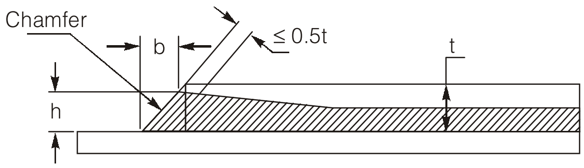
(b) in single and double bevel welds the sum of normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50 percent of the design shear stress.

10.5.10.2.2 Combined Bearing, Bending and Shear

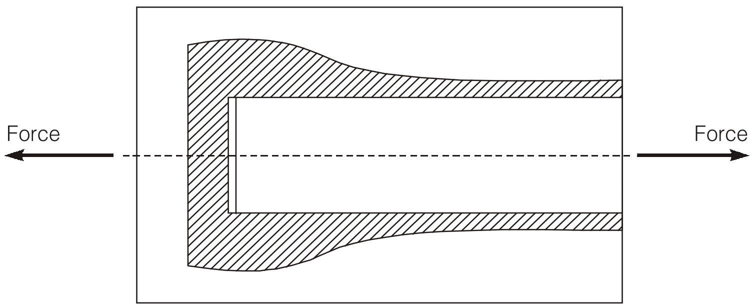
Where bearing stress, f_{br} is combined with bending (tensile or compressive), f_b and shear stresses, q under the most unfavourable conditions of loading in butt welds, the equivalent stress, f_e as obtained from the following formula, shall not exceed the values all allowed for the parent metal:

$$f_e = \sqrt{f_b^2 + f_{bf}^2 + f_b f_{br} + 3q^2}$$

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 and
q = shear stress, in N/mm^2



h : b = 1 : 2 or flatter



End fillet weld normal to direction or force

10.7 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:

1. Connections in rigid construction – a bending moment of at least 0.5 times the member design moment capacity.

2. Connections to beam in simple construction – a shear force of at least 0.15 times the member design share capacity or 40 kN, whichever is lesser.
3. Connections at the ends of tensile or compression member- a force of at least 0.3 times the member design capacity.
4. Splices in members subjected to axial tension – a force at least 0.3 times the member design capacity in tension.
5. Splices in members subjected to axial compression – for ends prepared for full contact, 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 0.3 times the member design capacity in axial compression.

   In addition, splices located between points of effective lateral support shall be designed for the design axial force, P_d plus and design bending moment, not less than the design bending moment $M_d = (P_d l_s)/1000$, where, l_s is the distance between points of effective lateral support.
6. 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.
7. 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 shall satisfy requirements in (4), (5) and (6) above, simultaneously. For earthquake load combinations, the design action effects specified in this section may need to be increased to meet the required behaviour of the steel frame.

SECTION 11 : WORKING STRESS DESIGN

11.1 GENERAL

11.1.1 General design requirements of Section 3 shall apply in this section.

11.1.2 Methods of structural analysis of Section 4 shall also apply to this section. The elastic analysis method shall be used in the working stress design.

11.1.3 The working stress shall be calculated applying respective partial load factor for service load/working load.

11.1.4 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 increase shall be limited to 25 percent. Such an increase in allowable stresses should not be considered if the wind or seismic load is the major load in the load combination (such as acting along with dead load alone).

11.2 TENSION MEMBERS

11.2.1 Actual Tensile Stress

The actual tensile stress, f_t on the gross area of cross-section, A_g of plates, 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,

$$f_t = \frac{T_s}{A_g}$$

The permissible stress, f_{at} is smallest 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 under working (service) load
A_g = gross area
T_{dn} = design strength in tension of respective plate/angel calculated in accordance with **6.3**, and
T_{db} = design block shear strength in tension of respective plate/angle calculated in accordance with **6.4**.

11.3 COMPRESSION MEMBERS

11.3.1 Actual Compression Stress

The actual compressive stress, f_c at working (service) load, P_s of a compression member shall be less than or equal to the permissible compressive stress, f_{ac} as given below:
Actual compressive stress,

$$f_c = \frac{P_s}{A_e}$$

The permissible compressive stress,

$$f_{ac} = 0.60\, f_{cd}$$

where,

A_e = effective sectional area as defined in **7.3.2**, and
f_{cd} = design compressive design stress as defined in **7.1.2.1**.

11.3.2 Design Details

Details of the compression members shall confirm to **7.3**.

11.3.3 Column Bases

The provisions of **7.4** 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 compression

a, b = larger and smaller projection of the slab base beyond the rectangle circumscribing the column, respectively; and

f_{bs} = permissible bending stress in column base equal to $0.75\ f_y$

11.3.4 Angle Struts

Provisions of **7.5** shall be used for design of angle struts, except that the limiting actual stresses shall be calculated in accordance with **11.3.1**.

11.3.5 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 given in **11.3.1**.

11.4 MEMBERS SUBJECTED TO BENDING

11.4.1 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:

$$f_{bc} = \frac{M_s}{Z_{ec}} \text{ and } f_{bt} = \frac{M_s}{Z_{et}}$$

The permissible bending stresses, f_{abc} or f_{abt} shall be smaller of the values obtained from the following:

(a) Laterally supported beams and beams bending about the minor axis:

1. Plastic and compact sections

$$f_{abc} \text{ or } f_{abt} = 0.66\ f_y$$

2. Semi-compact sections

$$f_{abc} \text{ or } f_{abt} = 0.60\ f_y$$

(b) Laterally unsupported beams subjected to major axis bending:

$$f_{abc} = 0.60\ M_d/Z_{ec}$$
$$f_{abt} = 0.60\ M_d/Z_{et}$$

(c) Plates and solid rectangle 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 extreme compression and tension fibre, 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**.

11.4.2 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, $$\tau_b = \frac{V_s}{A_v}$$

The permissible shear stress is given by

(a) When subjected pure shear

$$\tau_{ab} = 0.40\, f_y$$

(b) When subject to shear buckling

$$\tau_{ab} = \frac{0.70\, V_a}{A_v}$$

where, V_n = design shear strength given in **8.4.2.2** (a) and
A_v = shear area of the cross-section as given in **8.4.1**.

Example:

Q.1 The maximum shear force at a section is 56 kN. An ISWB of height 350 mm, breadth 200 mm, thickness of web 8 mm, with a section modulus of 887 cm^3 is used as a beam at the section. The shearing stress is

(a) 10 N/mm^2 (b) 20 N/mm^2
(c) 28.4 N/mm^2 (d) 41.6 N/mm^2

[IES : 2001]

Ans. (b)

11.4.3 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**.

11.6 CONNECTIONS

11.6.1 All design provisions of Section 10, except for the actual and permissible stress calculations, shall apply.

11.6.2 Actual Stresses in Fasteners

11.6.2.1 Actual stress in bolt in shear, f_{sb} should be less than or equal to permissible stress of the bolt, f_{asb} as given below.

The actual stress in bolt in shear,

$$f_{sb} = \frac{V_{sb}}{A_{sb}}$$

The permissible stress in bolt in shear,

$$f_{asb} = \frac{0.60\, V_{nsb}}{A_{sb}}$$

where, V_{sb} = actual shear force under working (service) load
V_{nsb} = nominal shear capacity of the bolt as given in **10.3.3** and
A_{sb} = nominal plain shank area of the bolt

11.6.2.2 Actual stress of 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,

$$f_{apb} = \frac{0.60\ V_{npb}}{A_{pb}}$$

where, V_{npb} = nominal bearing capacity of a bolt on any plate as given in **10.3.4**, and

A_{pb} = nominal bearing area of the bolt on any plate

11.6.2.3 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,

$$f_{tb} = \frac{T_s}{A_{sb}}$$

The permissible tensile stress of the bolt,

$$f_{atb} = \frac{0.60\ T_{nb}}{A_{sb}}$$

where, T_s = tension in bolt under working (service) load,

T_{nb} = design tensile capacity of a bolt as given in **10.3.5**, and

A_{sb} = nominal plain shank area of the bolt

11.6.2.4 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**

11.6.2.5 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:

$$\left[\frac{f_{sb}}{f_{asb}}\right]^2 + \left[\frac{f_{tb}}{f_{atb}}\right]^2 \le 1.0$$

where, f_{sb}, f_{tb} = actual shear and tensile stresses respectively, and

f_{asb}, f_{atb} = permissible shear and tensile stresses respectively

11.6.3 Stresses in Welds

11.6.3.1 Actual stresses in the throat area of fillet welds shall be less than or equal to permissible stresses, f_{aw} as given below:

$$f_{aw} = 0.4\ f_y$$

11.6.3.2 Actual stresses in the butt welds shall be less than the permissible stress as governed by the parent metal welded together.

❍❍❍❍