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Structural Steel Design

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Civil Services Examination

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Structural Steel Design

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01
CHAPTER

Basic Concepts

1.1 INTRODUCTION

Steel structure are made from organised combination of structural steel members to carry loads and provide adequate rigidity. Steel structure involve a sub-structure or members in a building made from structural steel. Steel structure facilitate ease of fabrication and faster erection of structure-Bolts and welding employed for joining. Depending upon the orientation of the member in the structure and its structural use, the member is subjected to forces either axial, shear, bending or torsion or a combination. Steel as a building material used extensively in various types of structure.

1.2 STEEL AS A STRUCTURAL MATERIAL

Advantages of steel as a structural material:

(a) As compared to other structural materials steel has high strength to weight ratio. It implies, steel possess very high strength and results in smaller sections as compared to other structural materials. Thus steel is particularly useful for carrying heavy loads with relatively small sections.

(b) If joints are taken care, it is the best water and gas resistance structure. Hence can be used for making water tank.

(c) Another important property of steel is that it is ductile. Due to this very useful property it does not fail abruptly but gives ample warning by yielding before actual collapse of the structure.

(d) Steel possess very high strength due to which it does not undergo fracture under large deformation and erection stresses.

(e) Steel has a very long life when maintained properly.

(f) Retrofitting of steel structures is quite easy as compared to other materials like RCC, timber, mortar etc.

(g) The resale value of steel is also very high amongst all building materials. Moreover, steel can be reused also.

(h) Steel is ultimate recyclable material.

(i) They can be erected at a fast rate.

(j) Addition and alteration can be made easily to steel structures.

(k) The properties of steel mostly do not change with time. This makes steel most suitable material for a structure.

(l) Being light, steel members can be conveniently handled and transported. For this reason, prefabricated members can be frequently provided.

Disadvantages of steel as a structural material:

(a) Steel when placed in exposed conditions is prone to corrosion. Thus steel structures require proper protection measures to be adopted right from its manufacturing.

(b) Steel is prone to fire and its strength reduces considerably due to high temperatures and thus steel structures require separate fire-proof treatment which ultimately adds to the cost of the structure. Almost from 600-700°C, half of steel strength reduced.

(c) Steel structures when subjected to cyclic loading (like turbo-generators of power plants etc.) and reversal of stresses undergo fatigue, this fatigue results in reduction of the strength of the steel.

(d) Under certain conditions particularly at locations of stress concentrations, steel may loose its ductility which gets enhanced at low temperatures and under fatigue loading. If steel loses its ductility property, then chances of brittle fracture increases.

(e) In steel construction, the designer is not having too many options as in RCC as far as size of the section is concerned. The designer is compelled to use the available standard rolled sections.

(f) If there are very large variations in tensile stress than this lead steel to be more in tension. Due to which steel tensile properties graph falls down.

(g) Maintenance cost of the steel structure is very high. Due to rusting action, expensive paints are require to renew time to time. So that, resistance against severe conditions increases.

(h) Steel member are costly compared to other material like RCC etc.

1.2.1 Rolled steel section

Selection of standard cross-sectional shape that is available rather than need the fabrication of a shape with unique properties and dimension. Hot rolling is the process for production for largest categories of standard shape. Rolling of hot steel allow deformation without any loss of ductility. During cooling, the variation in the cooling rates produces residual stresses which may be removed by subsequent strengthening processes.

Structural steel can be rolled into various sizes and shapes. Various types of steel shapes rolled are described as follows:

(i) Rolled steel I-sections **Fig. (a)**

(ii) Rolled steel channel section **Fig. (b)**

(iii) Rolled steel T-section **Fig. (c)**

(iv) Rolled steel angle section **Fig. (d)**

(v) Rolled steel tube section **Fig. (e)**

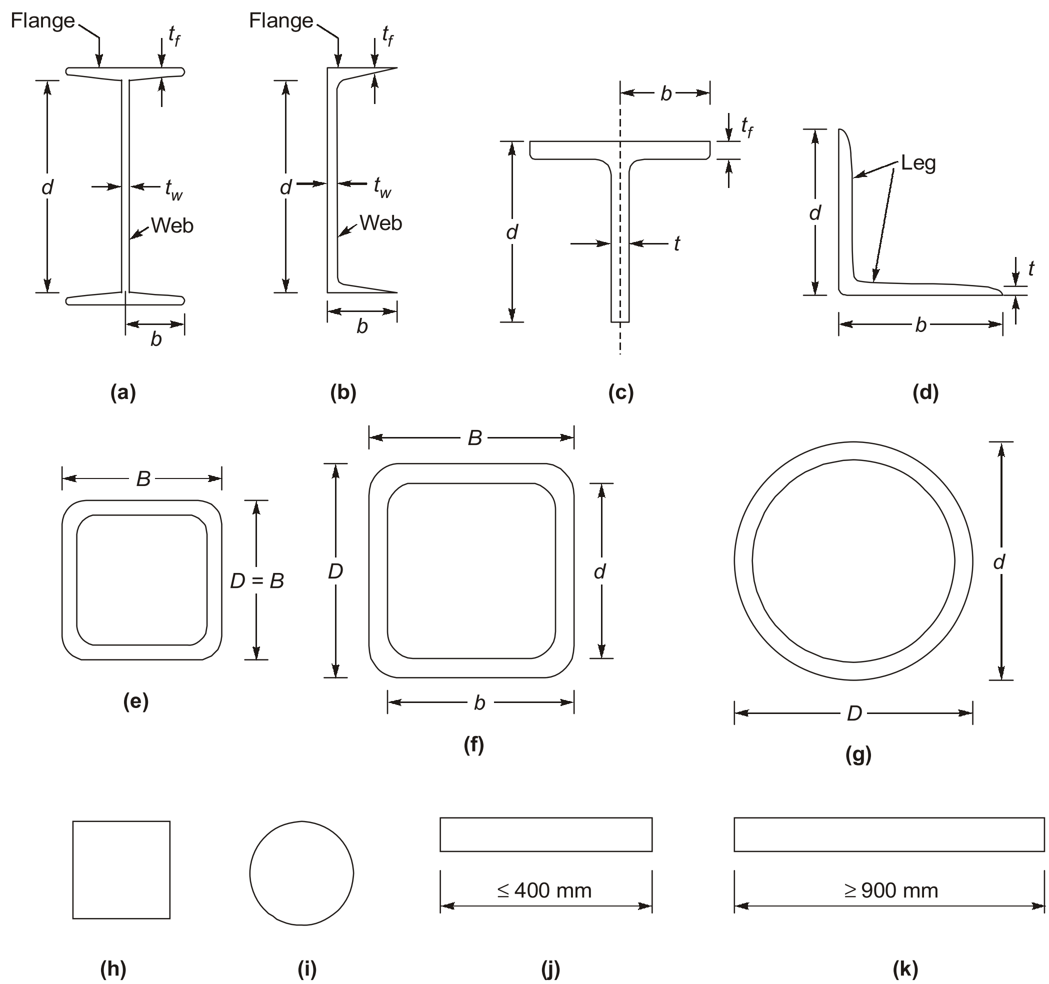
(vi) Rolled steel bars **Fig. (f)**

(vii) Rolled steel flats **Fig. (g)**

(viii) Rolled steel plates **Fig. (h)**

(ix) Rolled steel sheets

(x) Rolled steel strips



1.2.2 Sign Convention for Member Axes

x-x Longitudinal axis i.e. axis along the member

y-y Axis in the plane of the cross section which is:

(a) normal to the flanges [Fig. (a)]

(b) normal to the smaller leg in angle sections [Fig. (b)]

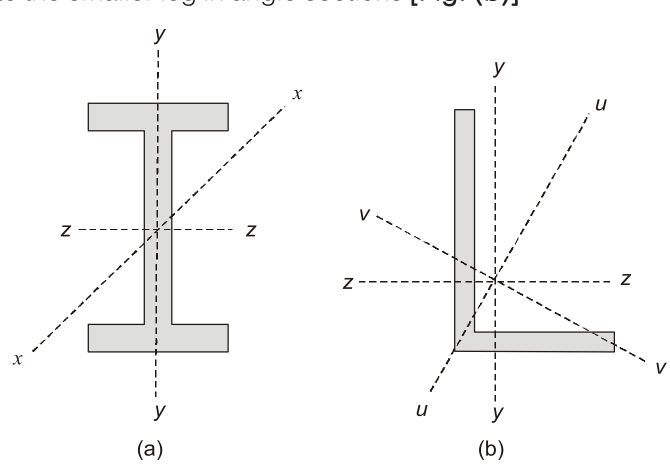


Fig. *Member axes notation*

z-z Axis in the plane of the cross section which is:

(a) parallel to the flanges [Fig. (a)]

(b) parallel to the smaller leg in angle sections [Fig. (b)]

u-u Major axis in case it does not coincide with the ***z-z*** axis [Fig. (b)]

v-v Minor axis in case it does not coincide with the ***y-y*** axis [Fig. (b)]

1.3 BUCKLING PHENOMENA

- Buckling is a phenomena, where a compression member is subjected to unwanted bending stresses due to unintended or accidental eccentricities of axial compression force. This bending stress causes the compression member to bend out of the axis leading to further increase in stress causing the member to ultimately fail.
- Local buckling is another related phenomena that is of concern usually in the design of steel structure. Local buckling takes place due to following conditions.
    - Rolled steel sections usually have outstanding flange elements that have unsupported edge and are thin and slender.
    - These sections are used for members such as columns, beams or beam columns in which at least a part of the section comes under compressor.
    - Due to localised compression and these thin and slender outstanding flange elements, localised out of plane buckling of these flange or web element occur.
    - Local buckling adversely affects the load carrying capacity of the section. Buckling affects the strength of section in the following ways:

      (a) Buckling may lead to overall failure of the structure rendering the plate element to be ineffective.

      (b) Buckling may give rise to redistribution of stresses and thus affects the load carrying capacity of the section.

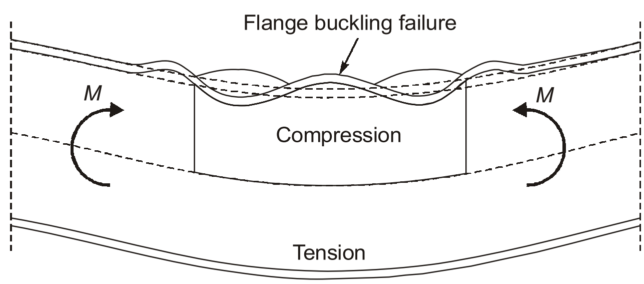


Fig. *Local flange buckling failure*

The effect of local buckling of plate element on the strength of the whole structure depends on the location of buckled element, its buckling and post buckling characteristics and the type of structural member.

1.3.1 Prevention of Local Buckling

The local buckling can be prevented by adopting a higher thickness value for the plate element. IS 800:2007 puts a limit on width to thickness ratio of component steel plates of the section as given in Table A.

Table A: Limiting width-to-thickness ratio

Compression Element			Ratio	Class of Section: Class 1 Plastic	Class 2 Compact	Class 3 Semi-Compact
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Comp. due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I-H-or box Section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative	d/t_w	$\frac{84\varepsilon}{1 + r_1}$ but $\leq 40\varepsilon$	$\frac{105\varepsilon}{1 + r_1}$	$\frac{126\varepsilon}{1 + 2\,r_2}$ but $\leq 40\varepsilon$
		If r_2 is negative	d/t_w		$\frac{105\varepsilon}{1 + 1.5r_1}$ but $\leq 40\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (both criteria should be satisfied)			b/t	9.4ε	10.5ε	15.7ε
			d/t	9.4ε	10.5ε	15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t	Not applicable		15.7ε
			d/t			15.7ε
			$(b + d)/t$			25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component						
Stem of a T-section, rolled or cut from a milled I or H-section			D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: (a) Moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
(b) axial compression			D/t	Not applicable		$88\varepsilon^2$
Circular hollow section	Compression due to bending		D/t	$42\varepsilon^2$	$52\varepsilon^2$	$88\varepsilon^2$
Mix-rolled rectangular hollow section (RHS)	Flange: compression due to bending		d/t	29.3ε	33.5ε	42ε
	Web: neutral axis at mid-depth		d/t	67.1ε	84ε	125.9ε
	Generally		d/t	$64\varepsilon/(1 + 0.6\,r_1)$ but $< 40\varepsilon$	$84\varepsilon/(1+r_1)$ but $< 40\varepsilon$	$125.9\varepsilon/(1 + 2\,r_1)$ but $< 40\varepsilon$

where, $\epsilon = \sqrt{\frac{250}{f_y}}$

> **NOTE**
> IS 800:2007 specifies no limit on minimum thickness requirement of steel sections but however a minimum thickness of 6 mm for main members and 5 mm for secondary members must be used in steel design and construction. This minimum thickness is required for better performance under adverse environmental conditions. In addition to that, steel in contact with water and soil and those subjected to alternate wetting and drying, an additional thickness of 1.5 mm should be provided.

- **Internal Elements:** These are the elements which are attached along both the longitudinal edges to other elements or to the longitudinal stiffeners connected at adequate intervals to the transverse stiffeners e.g. web of I-section and flanges and web of box section.
- **Outstand Elements:** These are also called as outstands and are attached only to one of the longitudinal edges to an adjacent element and the other edge is being free to get displaced out of its plane e.g. flange overhang of an I-section and legs of an angle section.

1.4 CLASSIFICATION OF CROSS-SECTION

- The phenomenon of local buckling imposes a limit to the extent to which sections can be made thin walled.

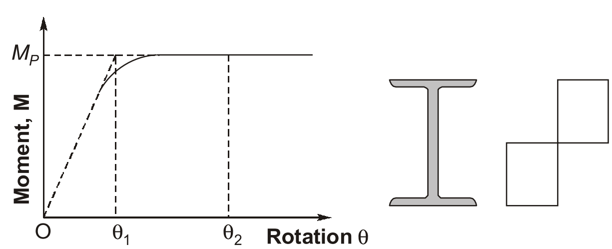


Fig. *Elastic-plastic moment rotation curve*

- The classification of cross-sections is done on the basis of moment rotation characteristics as shown in **Fig.** assuming that the flange or web plate does not buckle locally.

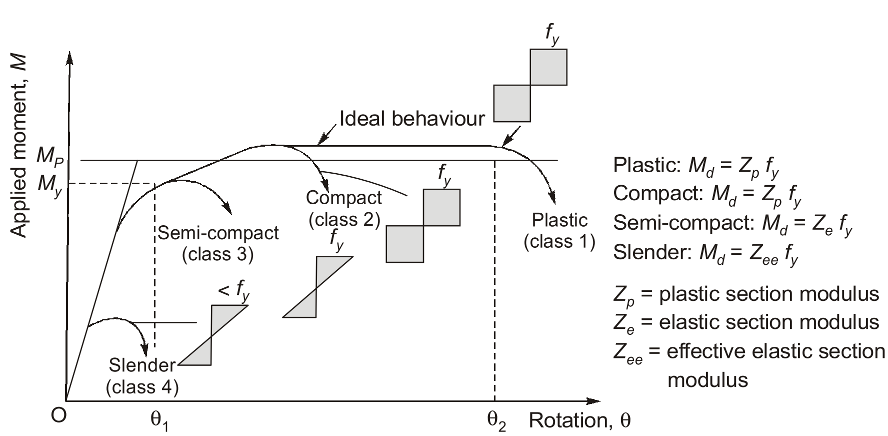


Fig. *Moment rotation characteristics of four classes of cross-section*

- It is essential to classify sections on the basis of their tendency to buckle locally before overall failure of member takes place.

1.4.1 Basis of Section Classification

(a) The plate elements of the cross section being thin may buckle locally about their weak axis especially due to compressive stresses. This local buckling can be avoided before a limit state is reached by limiting the width to thickness ratio of each element of the cross section subjected to compressive load, moment and shear.

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

(c) When elastic analysis is used, the member must be capable of developing the yield stress under tension and compression without undergoing local buckling.

Based on these considerations, four classes of sections are mentioned in **Table B.**

Table B: Cross-section classification and their characteristics

S.No.	Types of Sections	Characteristics
1.	Plastic (Class 1)	• Cross sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of a plastic mechanism, are called plastic sections. These sections are used in plastic analysis and design. • These sections are used in indeterminate frames forming plastic collapse mechanism. • The stress distribution for these sections is rectangular.
2.	Compact (Class 2)	• Cross sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism before buckling are referred to as compact sections. • These cross sections may develop full plastic stress distribution across the entire cross-section but do not have adequate ductility. • For a section to be compact, its compression elements must have width-to-thickness ratios equal to or less than the limiting values given in Table 1.1. These can be used for all the structural elements. • The stress distribution for these sections is rectangular.
3	Semi-compact (Class 3)	• Cross sections, in which the extreme fiber in compression can reach yield stress (assuming an elastic distribution of stress), but cannot develop the plastic moment of resistance due to local buckling are referred to as semi-compact sections. • The yield stress reaches only in some parts of compression elements before buckling occurs. • It is not capable of reaching a fully plastic stress distribution. • These sections are used in elastic design. The stress distribution for such sections is triangular.
4.	Slender (Class 4)	• Cross sections, in which the elements buckle locally even before the attainment of yield stress are classed as slender sections. • These sections are used in cold-formed members and do not comply with the requirements of Table 1.1. • The effective section for design should be calculated by deducting width of compression plate element in excess of the semi-compact section limit.

1.5 MODES OF FAILURE IN BEAM

(a) **Bending:** The gravity loading gives rise to bending of beam. This causes longitudinal stresses to be developed in the beam. As the bending moment increases, these flexural stresses increase further till they reach yield stress. At certain point, either the steel yields in tension and/or yields in compression. At this stage of loading, the beam section becomes plastic, fails by formation of a plastic hinge at the location of maximum moment being induced by the loading. **Fig.** shows the various stages in a beam bending till failure.

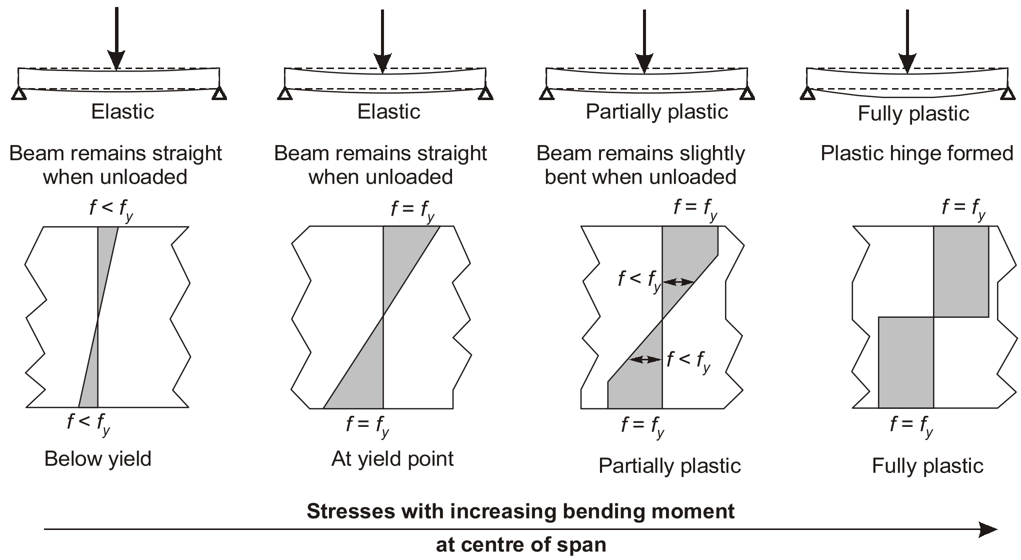


Fig. *Various stages of beam bending failure*

(b) **Shear:** Generally in portions near to the supports, very large shear exists. In this case the beam may fail in shear if the web is not of sufficient thickness. Formation of plastic hinges accompanies this process as shown in **Fig. (a).**

(c) **Shear buckling:** During the shearing process, if the web is too thin then it is prone to get fail by buckling (or rippling) as shown in **Fig. (b) & (c).**

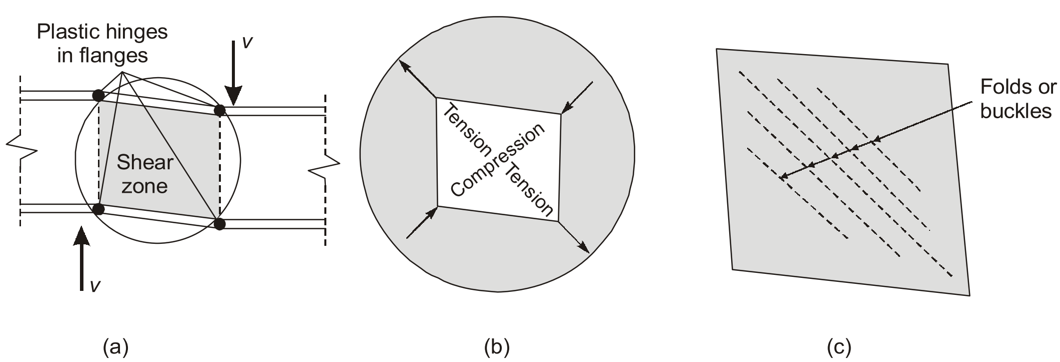


Fig. *Shear and shear buckling failure (a) shear failure, (b) shear buckling*

(d) **Web crushing and buckling:** Because very high vertical stresses act on supports and at location of point loads, the beam web may get fail by crushing or buckling as shown in **Fig.**

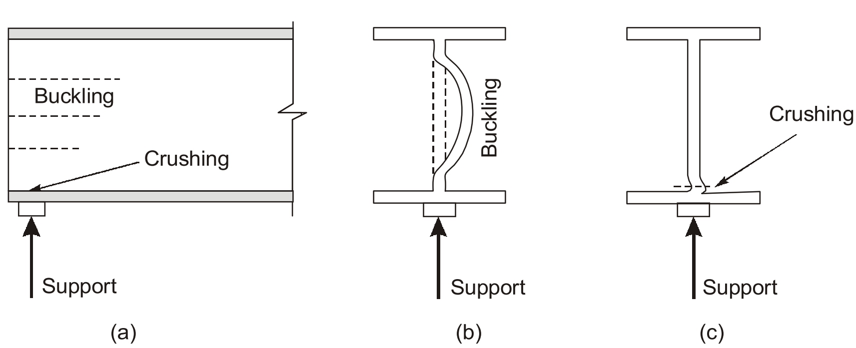


Fig. *Web buckling and web bearing failures*

(e) **Lateral torsional buckling:** This type of failure occurs when the beam has high flexural stiffness in the vertical plane (i.e. in the direction of gravity) as compared to the horizontal plane. The beam has a tendency to deflect sideways.

- When I-sections are used as beams or beam columns the compression flange is under compressive stress and has a tendency to buckle but it is attached to the tension flange which resist the buckling giving rise to torsion within the beam section. This torsion twist and warp the unrestrained part as beam leading to lateral torsional buckling. A nominal amount of torsional restraint is assumed to exist when web of the beam is connected through cleat angles, end plates etc. as shown in Fig.

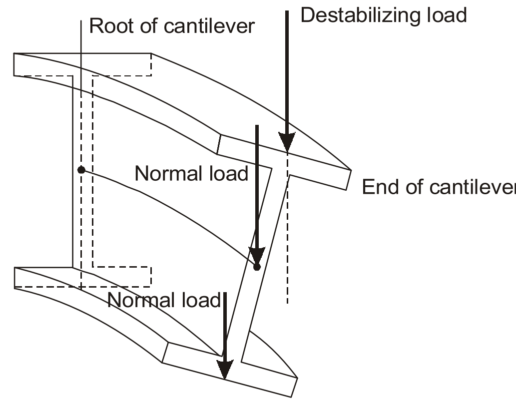


Fig. *Lateral torsional buckling of cantilever*

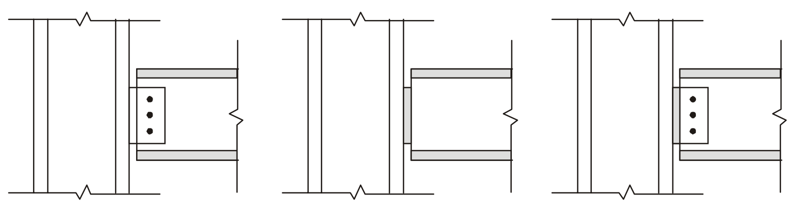


Fig. *Nominal torsional restraint at beam support supplied by (a) web cleats (b) end plate (c) fin plate*

(f) **Deflection:** It is quite practical that a beam may not get fail due to excessive deflection but on the other hand too much deflection can lead to discomfort to the occupants along with stripping off of flooring, finishes etc.

1.6 DESIGN OF STEEL STRUCTURES

- Errors and uncertainties get involved in precisely assessing the probable loads that will act on the structure and the structural behavior of the materials.
- The various structural design requirement criteria related to corresponding limit states and thus the design of structure to comply with all the essential requirements may be called as limit state of design.
- There are two design criteria viz. the **strength design criteria** and the **stiffness design criteria.**

1.6.1 Strength Design Criteria

- These are related to possible modes of failure of a structure due to over loading and/or understrength conditions.
- These design criteria are concerned with yielding, buckling, fatigue, brittle fracture corrosion.
- The major design philosophies under these criteria are the **working stress method, ultimate load method** and the **limit state method.**

1.6.2 Stiffness Design Criteria

- These design criteria are related to serviceability of the structure under working loads and are prominently concerned with ensuring that structure has sufficient stiffness that may otherwise lead to excessive deformations including deflections, distortion, sagging etc.

1.7 LIMIT STATE DESIGN

- In the limit state design of structures, all conditions are taken into account that makes the structure unfit for use.
- This limit state method of design considers all the relevant conditions pertaining to limit states of strength and serviceability.
- The strength limit states are based on load carrying capacity of the structure which includes buckling, fatigue, fracture etc. Serviceability limit states are based on performance of the structure under the actual application of service loads and includes deflection, vibration, corrosion, ponding etc.
- The loads at working conditions are factored by the use of partial safety factors for loads resulting in factored loads/design loads.
- The normal strength of the material is called as its ultimate capacity and the corresponding design strength of the material is obtained by dividing the ultimate capacity with the partial safety factor. Thus the member so designed must meet the basic criteria as

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

REMEMBER The section designed must satisfy the serviceability criteria over and above the strength criteria.

1.7.1 Advantages of Limit State Design

(a) This method recognizes that design parameters are variants.
(b) This method logically deals with the fact that there exists always a possibility of variations in loads and material properties.
(c) This method gives different weightages to different loads and materials
(d) Limit state method of design is far better than other design philosophies.

1.7.2 Disadvantages of Limit State Design

- There is a likelihood of design errors because of more complex theory which are not so in the working stress method of design.

1.8 ACTIONS (LOADS)

The actions to be considered in a design are:

(1) Direct actions experienced by the structure due to self weight and external actions.
(2) Imposed deformations such as that due to temperature and settlement.

IS 800-2007, classified various actions in the following three groups:

(a) **Permanent Actions (Q_p):** Actions due to self weight and fixed equipment etc.
(b) **Variable Actions (Q_v):** Actions during construction and service stage such as imposed loads, wind loads and earthquake loads etc.
(c) **Accidental Actions (Q_n):** Actions expected due to explosions and impact of vehicles etc.

1.8.1 Characteristic Action (Q_c)

The characteristic actions (Q_c) are defined as the values of different actions which are not expected to be exceeded with more than 5 percent probability, during the life of the structure. One can work out these actions by statistical analysis, in all special cases, subjected to minimum values specified in codes.

In an absence of statistical analysis, the loads presented in IS 875 and other special codes may be considered characteristic loads.

1.8.2 Design Actions

Noting the importance of safety in civil engineering structures and the uncertainties involved in the analysis, design and construction, code specifies taking design actions as partial safety factor times the characteristic actions. The partial safety factors specified by code for limit state of strength and serviceability differ. The partial safety factors for loads are as given in Table C and design load Q_d is to be found as

$$Q_d = \sum_k \gamma_{fk} Q_{ck}$$

where γ_{fk} is partial safety factor for k^{th} load.

Table C: Partial safety factors for loads, γ_f for limit state [Table 4 of IS 800 - 2007]

Combination	Limit State of Strength					Limit State of Strength			
	DL	LL		WL/EL	AL	DL	LL		WL/EL
		Leading	Accompanying				Leading	Accompanying	
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 (0.9)	-	-	1.5	-	1.0	-	-	1.0
DL + ER	1.2 (0.9)	1.2	-	-	-	-	-	-	-
DL + LL + AL	1.0	0.35	0.35	-	1.0	-	-	-	-

NOTE

- Lower value of γ_f for DL is to be considered if DL causes higher value for load effect and lower value is to be considered, if DL contributes to the stability of structure against overturning while designing for stability.
- DL = Dead Load, LL = Imposed Load, WL = Wind Load, CL = Crane Load, AL = Accidental Load, ER = Erection Load and EL = Earthquake Load.

1.9 DESIGN STRENGTH

In using the strength value of a material for design, the following uncertainties should be accounted:

(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 materials.

Hence IS 800 - 2007, recommends reduction in strength of materials by a partial safety factor γ_m which is defined as

$$\gamma_m = \frac{S_u}{S_d} \quad \text{i.e.,} \quad S_d = \frac{S_u}{\gamma_m}$$

Where, S_u = ultimate strength and S_d = design strength.

These value are as shown in table D.

1.10 DESIGN LIMITS

Deflection limits are specified from the consideration that those do not cause damage to finishing. Deflections are to be checked to adverse but realistic combination of service loads and their arrangement.

Table D: Partial safety factors for materials γ_m [Table E of IS 800 - 2007]

S.No.	Definitions	Partial Safety Factor	
1.	Resistance, governed by yielding (γ_{mo})	1.10	
2.	Resistance of member to buckling (γ_{mo})	1.10	
3.	Resistance governed by ultimate stress (γ_{m1})	1.25	
4.	Resistance of connections	**Shop fabrication**	**Field fabrication**
	(a) Bolds-friction type γ_{mf}	1.25	1.25
	(b) Bolds-bearing type γ_{mb}	1.25	1.25
	(c) Rivets γ_{mr}	1.25	1.25
	(d) Welds γ_{mw}	1.25	1.50

Elastic analysis may be used to find deflection. Design loads for this purpose is the same as characteristic load (i.e., partial safety factor γ_f = 1.0) except when apart from DL, LL, CL and some more imposed loads are considered (refer table E)

The deflection limits specified by IS 800:2007 are as shown in below table E.

1.11 OTHER SERVICEABILITY LIMITS

Apart from deflection requirement, the design should also satisfy the following serviceability units:

(a) Vibration limit

(b) Durability consideration

(c) Fire resistance

1.11.1 Vibration Limit

Though most of the structures are designed for strength and then checked for deflection limits, some of the structures need check for vibration limits. The structures the floors of which support machineries, the flexible structures (with height of effective width ratio exceeding 5 : 1) etc, should be investigated for vibration under dynamic loads. In such cases there are possibilities of resonance, fatigue failures. IS 800 - 2007 gives a set of guidelines to take care of vibration limits in its Annex C.

1.11.2 Durability Considerations

The following factors affect the durability of a steel structure:

(a) Environment

(b) Degree of exposure

(c) Shape of the member and the structural detail

(d) Protective measures

(e) Ease of maintenance

Table E: Deflection Limits [Table 6 in IS 800]

Type of building (1)	Deflection (2)	Design Load (3)	Member (4)	Supporting (5)	Maximum Deflection (6)
Industrial Building	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter Supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50*t*)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50*t*)	Gantry	Crane	Span/1000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane + wind	Column/ frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Building	**Vertical**	Live load	Floor and Roof	Elements not susceptible to cracking	Span/300
				Elements susceptible to cracking	Span/360
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180
	Lateral	Wind	Building	Elastic cladding	Height/300
				Brittle cladding	Height/500
		Wind	Inter storey drift		Storey height/300

A designer should refer to the IS code provisions given in section 15 of 800-2007 and also to specialised literature on durability.

1.11.3 Fire Resistance

A steel structure should have sufficient fire resistance level (FRL) specified in terms of minutes depending upon the purpose for which the structure is used and the time taken to evacuate in case of fire. For detailed specifications a designer may refer section 16 of IS 800-2007 along with IS 1641, IS 1642, IS 1643 and any other specialised literature on fire resistance.

1.12 STABILITY CHECKS

After designing a structure for strength and stability, it should be checked for instability due to overturning, uplift of sliding under factored loads. In checking for instability disturbing forces should be taken as design loads and stabilising forces may be taken as design loads (factored loads) with lesser factor of safety (0.9) as specified in previous table C.

A structure should be adequately stiff against sway and fatigue also.

In the chapters to follow now onwards, design principles are made clear from the point of limit states of strength and deflections. In most of the buildings these are the predominant limit states, but in all important and special buildings, a designer has to ensure that other limit states are not exceeded.

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