Code Of Practice On Steel Structures - A Review Of IS 800: 2007

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Codes of practice reflect the combined wisdom of a profession. They provide the minimum requirements that a design has to satisfy. They have a legal status, in the sense that any failure consequent to the violation of the provisions of a code can land a designer into a legal liability. However, the codes of practice do not prescribe a process of design nor they constrain the designer from adopting the latest knowledge and developments in the profession to do the design. More recently, the provisions of the codes of practice have been incorporated in the computer aided design software to ensure the design meets the requirements of the governing code. Hence, the method of representing the code in such software can greatly affect the transparency and longevity of the software. With continuing research and development activities at an accelerated pace, there is great explosion in the generation and dissemination of knowledge in all areas of engineering. This has led to the need to revise the governing codes of practice at regular intervals, in order to reflect the more recently generated professional knowledge appropriately in the corresponding codes.

In India, the Bureau of Indian Standards (BIS) is the statutory body that publishes the codes of practice to be followed in the Indian Professional practice. Though the codes of practices of other countries such as USA are revised at regular intervals, the codes issued by BIS are revised only after 20 to 25 years. The second revision of IS 800 was published in 1984. The third revision of the code was released after about 24 years, in Feb 2007, by the BIS[1]. The material contained in the code reflects the state-of-the-art of knowledge, and is based on the provisions in other international codes as well as other research publications. The clauses contained in the code were developed by a team headed by Prof. R. Narayanan and later by Prof. Kalyanaraman of IIT Madras[2]. This version of the code is based on the limits state method of design philosophy whereas the earlier version was based on working stress method. This article reviews some of the important provisions of the new code.

Codal Provisions

The code is divided into the following 17 Sections. It also contains seven appendices.

1. General
2. Materials
3. General Design Requirements
4. Methods of Structural Analysis
5. Limit State Design
6. Design of Tension Members
7. Design of Compression Members
8. Design of Members subjected to Bending
9. Member subjected to combined forces
10. Connections
11. Working Stress Design
12. Design and Detailing for Earthquake Loads
13. Fatigue
14. Design Assisted by Testing
15. Durability
16. Fire Resistance
17. Fabrication and Erection

Comparing the provisions of the 1984 version of the code with that of the present code, it is seen that the present code contains major revisions. It gives a separate chapter on Methods of Structural Analysis, which discusses the following methods of analysis[3, 4].

First-order elastic analysis
- Second-order elastic analysis (includes methods to consider the effect of connection flexibility)
- Linear buckling analysis
- Inelastic buckling analysis
- First-order plastic analysis

Second-order inelastic analysis
- Plastic zone method
- Elastic plastic hinge method
- Refined plastic hinge method
- Notional load plastic hinge method
- Quasi-plastic hinge method

The second-order inelastic methods are often referred to as advanced analysis methods. This means that they take into

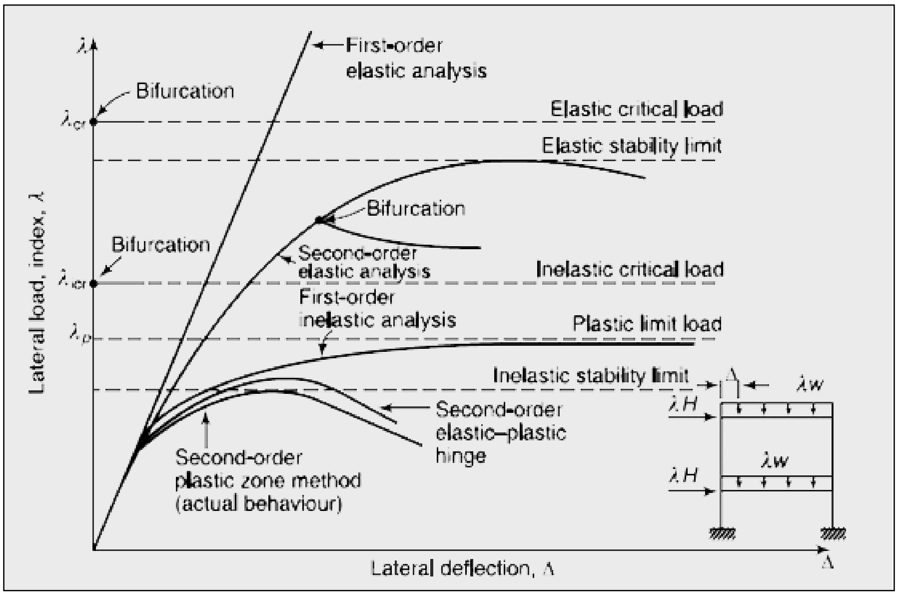


Fig.1 Load-Displacement Characteristics Of Different Methods Of Analysis

account the relevant material properties, residual stresses, geometric imperfections, second-order effects, three-dimensional effects, erection procedures, and interaction with the foundations. Thus, the advance analysis methods incorporate both strength and stability behaviour in such a way that separate members design is not required. They directly assess the strength and stability of the overall system, including the interaction of the member strength and stability. In addition to IS: 800, Eurocode 3 (EC3 1992)[5], Canadian Code (CSA-S16.1 2001)[6], Australian Code (AS-4100 1998)[7],and the American code (ANSI/AISC 360-05)[8], permit the use of advanced analysis methods, which eliminate the tedious and sometimes confusing member capacity checks in the conventional limit states method. This often leads to significant savings in design. The code also gives expressions for modeling various semi-rigid connections.

The main shift is from working stress design to limits states design (It is of interest to note that the concrete code IS 456: 2000 has adopted limit states design and all the international codes on steel structures have adopted some form of limit states design). Though the code allows design using working stress method, it is relegated to the end of the code, thus discouraging the designers to use it. The code also gives provisions for design and detailing for earthquake loads (Note that the recently released earthquake resistant design code IS 1893 (Part I): 2002 gives guidelines mainly for reinforced concrete structures). However, the designers may also refer to the American code, ANSI/AISC 341-05, for more detailed earthquake resistant design provisions[9].

The code for the first time has introduced provisions for fatigue (fatigue provisions are important for structures subjected to alternating loads, such as bridges, cranes, and structures supporting machinery), durability (though older codes gave importance to strength and stability, durability has become one of the main factors for design due to the early deterioration of recently built structures and also due to the dwindling natural resources. It is interesting to note that there are no clauses on minimum thickness of members) and fire resistance (fire resistance has become an important factor in the design due to the recent fire accidents in several multi-storey building and subsequent loss of life). Since it is difficult to review all these changes in a short paper, only a few important provisions are discussed.

Limit States Design

The fundamental requirement of a structural design is that the elements of the structure should have adequate and reliable safety against failure, the structure should remain serviceable during its intended use, and the design is economical. At the design stage, there are uncertainties about several factors affecting safety. Some of these factors are:

- The uncertainty in predicting loads which will be experienced during the lifetime of a structure due to random variation of loads (This uncertainty is further enhanced due to accidental loads, such as the recent plane attack on World Trade Center, USA[10] and the terrorist attacks on several buildings throughout the world[11]).
- The variations of strengths between nominally identical structural elements, due to design assumptions, random variation in the material strengths and the member dimensions.
- The consequence of mistakes and errors made by people involved in design and construction.

One of the main objectives in most codes is to ensure safety of the general public. For example, the IS 800:2007 should ensure that steel structures designed based on its provisions are safe. The term safe is nebulous to define. Over the years, there has been considerable change in the concept of safety and the approach taken by the design codes to ensure safety.

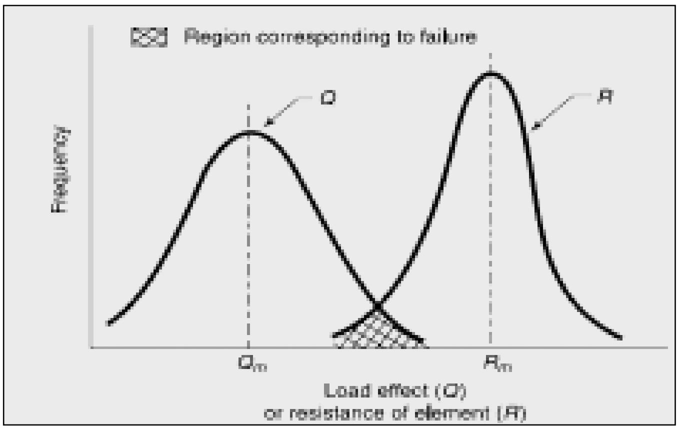


Fig. 2 Pictorial Representation Of The Variability Of Loads And Strength

No structure is fail-safe, since both loads and member strength are random variables. This is reflected in Fig. 2. There is always a probability, however small, that the actual load on a structure exceeds the strength of the structure[12]. The objective of the codes of practice has been to keep the probability of failure below an acceptable low level (10^4 to 10^5).

As we design a structure with very low probability of failure, the cost of the structure increases simultaneously. Attempting to design a highly safe structure (say, a low probability of failure of about 10^{10}) may increase the cost of the structure to a level that an individual or the society cannot afford to pay. On the other hand, designing for a higher probability of failure could lead to considerable cost to the individual or society in terms of the consequences of a failure. Thus, the design becomes a balancing act between safety and cost. Suitable values for partial safety factors are adopted in the code to take care of the reliability of design[13]. The limit states considered in the code may be grouped into the following two types:

- Ultimate (safety) limit states, which deal with strength, sway or overturning, sliding, buckling, fatigue fracture and brittle fracture.
- Serviceability limit states, which deal with discomfort to occupancy and/ or malfunction, caused by excessive deflection, vibration, corrosion (and subsequent loss of durability), fire resistance, etc.

Partial Safety Factors

The variation due to the difference between the overall resistances of a structure to a set of loads, predicted by the

Table 1: Partial Safety Factors For Loads, γ_{fk}, for Limit States[1]

Combination	Limit State of Strength					Limit State of Serviceability			WL/EL
	DL	LL		WL/EL	AL	DL	LL		
		Leading	Accompanying (CL, SL etc.)				Leading	Accompanying (CL, SL etc.)	
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	-	-	-		

*This value is to be considered when stability against overturning or stress reversal is critical. Abbreviations: DL = Dead Load, LL = Imposed Load (Live Loads), WL = Wind Load, SL = Snow Load, CL = Crane Load (Vertical / horizontal), AL = Accidental Load, ER = Erection Load, EL = Earthquake Load.

Note: The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis.

design calculations and the resistance of the actual structure is taken care of by a set of partial safety factors or γ factors. The specific effect of variability in material and geometric properties is taken care off by the partial safety factors for strength, γ_m. The variability of the loads on the structure, or more specifically, the load effects on the various structural components, is reflected through the partial safety factors for loads (load factors) γ_{fk}. For a safe structure,

Design Action = Design strength . . . (1)

The design actions, Qd, is expressed by

$Q_d = \Sigma \gamma_{fk} Q_{ck}$ . . . (2)

Where γ_{fk} = partial safety factor for different loads k, as given in Table 1.

The design strength, S_d is given by

Design strength, S_d = Theoretical ultimate strength, S_u / γ_m . . . (3)

Where, γ_m is taken as given in Table 2.

The code also specifies deflections limits for vertical and lateral loads for industrial buildings and other buildings (see Table 6 of IS 800:2007[1]).

Classification Of Cross-Sections

Determining the resistance (strength) of structural steel components requires the designer to consider first the cross-sectional behaviour and second the overall member behaviour - whether in the elastic or inelastic material range, cross-sectional resistance and rotation capacity are limited by the effects of local buckling[14].

In the code cross sections are placed into four behavioural classes depending upon the material yield strength, the width-to-thickness ratios of the individual components (e.g., webs and flanges) within the cross section, and the loading arrangement. The four classes of sections are defined as follows (see also Fig.3):

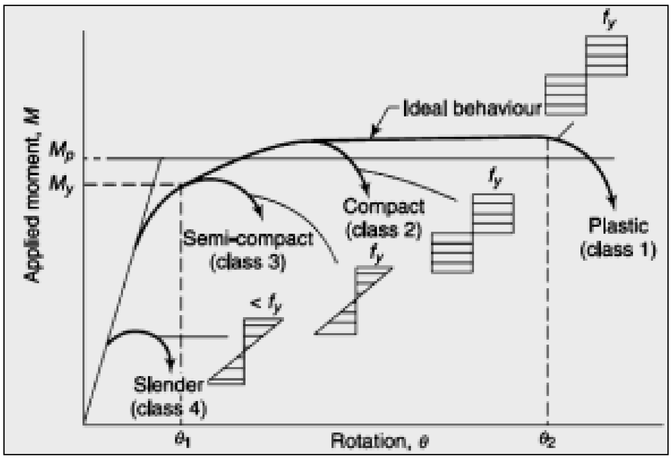


Fig.3 Moment-Rotation Behaviour Of The Four Classes Of Cross-Sections As Defined By IS 800: 2007

(a) Plastic or class 1 Cross sections which can develop plastic hinges and have the rotation capacity required for the failure of the structure by the formation of a plastic mechanism (only these sections are used in plastic analysis and design).

(b) Compact or class 2 Cross sections which can develop their plastic moment resistance, but have inadequate plastic hinge rotation capacity because of local buckling.

(c) Semi-compact or class 3 Cross sections in which the elastically calculated stress in the extreme compression fibre of the steel member, assuming an elastic distribution of stresses, can reach the yield strength, but local buckling is liable to prevent the development of the plastic moment resistance.

(d) Slender or class 4 Cross sections in which local buckling will occur even before the attainment of yield stress in one or more parts of the cross section. In such cases, the effective sections for design are calculated by deducting the width of the compression plate element in excess of the semi-compact section limit.

It has to be noted that only plastic sections should be used in indeterminate frames forming plastic-collapse mechanisms. In elastic design, semi-compact sections can be used with the understanding that the maximum stress reached will be M_y. Slender sections also have stiffness problems and are not preferable for hot-rolled structural steelwork. Compact or plastic sections are used for compression members, since they have more stiffness than semi-compact or slender members[3].

Table 2: Partial Safety Factor for Materials[1], γ_m

Sl. No	Definition		Partial Safety Factor
1.	Resistance, governed by yielding, γ_{m0}		1.10
2.	Resistance of member to buckling, γ_{m0}		1.10
3.	Resistance governed by ultimate stress, γ_{m1}		1.25
4.	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

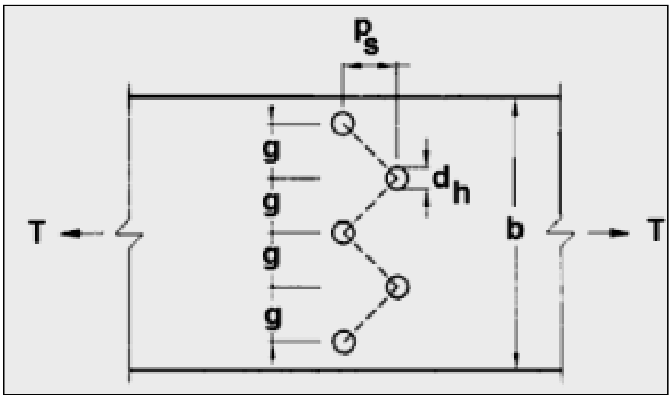


Fig.4 Plate With Staggered Holes

The maximum value of limiting width-thickness ratio of different classifications of sections is given by the code (See Table 2 of code). When different elements of a cross-section fall under different classifications, the most critical one has to be selected to represent the classification of the cross-section. Most of the hot-rolled sections available in the market fall under the category of plastic or compact sections[3].

Design Of Tension Members

The factored design tension T, in the members shall satisfy the following requirement.

$T < T_d$ . . . (4)

Where T_d = design strength of the member under axial tension T_d is the lowest of the design strength due to the yielding of cross-section, T_{dg}, rupture of critical section T_{dn} and block shear failure, T_{db}.

Design Strength Due To Yielding Of Gross-Section

Tension yielding of the members at the gross cross-section is given by $T_{dg} = f_y A_g / \gamma_{m0}$ . . . (5a)

Where f_y is the yield stress of material in MPa, and A_g is the gross area of cross-section

Design Strength Due To Rupture Of Critical Section

Plates: Tension rupture of the plate at the net cross-section is given by

$T_{dn} = 0.9\ f_u\ A_n / \gamma_{m1}$ . . . (5b)

Where f_u = ultimate stress of the material in MPa, and A_n = Net effective area of the member given by (see Fig. 4 for the definition of variables)

$A_n = [b - nd_h + \Sigma \frac{p_{si}^2}{4g_i}]$ . . . (5c)

Angle Members

For angle members connected through one leg, the design rupture strength (T_{dn}) is calculated as:

$T_{dn} = 0.9\ f_u\ A_{nc} / \gamma_{m1} + \beta\ A_{go}\ f_y / \gamma_{m0}$ . . . (5d)

Where $\beta = 1.4 - 0.076\ (w/t)\ (f_y/f_u)\ (b_s/L_c)$

$= (f_u \gamma_{m0} / f_y\ \gamma_{m1}\ ) \geq 0.7$

Where w and b_s are as shown in Fig 5 and L_c is the length of the connection, taken as the distance between outermost bolts in the joint measured along the direction of load (length of weld in the case of welded connection).

For preliminary sizing, the rupture strength may be taken approximately as

$T_d = \alpha A_n f_u / \gamma_{m1}$ . . . (5e)

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

Strength As Governed By Block Shear Failure[15, 16, 17]

Block shear failure was recognized as a failure mode first in 1978, when Birkemoe and Gilmor conducted tests on coped beams with bolted web connections, and incorporated in AISC specifications in 1978 (Epstein and Aleksiewicz, 2008). Block shear failure in angles were investigated after the failure of Hartford Civic Center roof, Connecticut in 1978. Block shear failure in bolted / welded connections is characterized by a condition, where a "block" of material, in a pattern surrounding

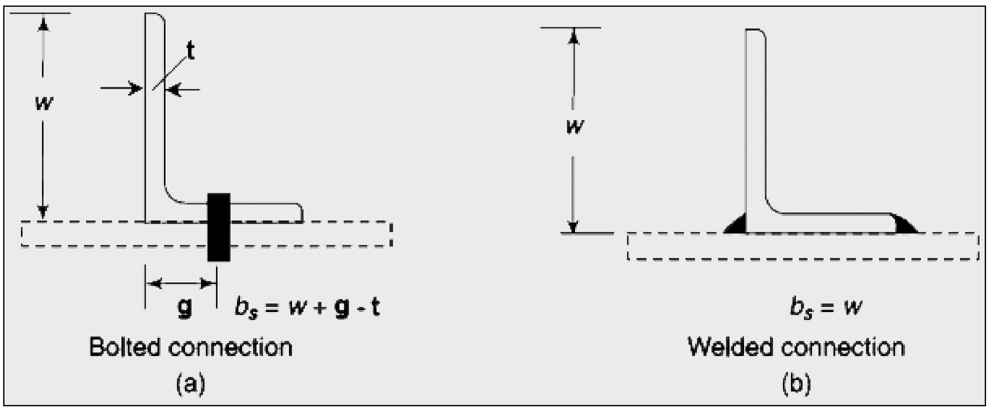


Fig. 5 Angles With Single Leg Connection

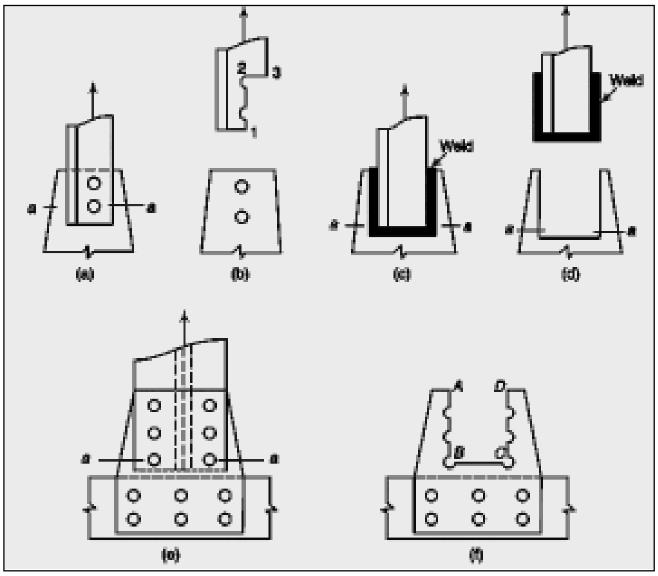


Fig. 6 Examples Of Block Shear Failures

the bolted region, reaches its capacity through a combination of tension and shear. If the connection is loaded further, the block is eventually displaced from the connection region (see Fig.6). Block shear is usually initiated with tension fracture. The block shear strength, Tdb of the connection shall be smaller of

$$T_{db} = A_{vg}\, f_y / (\sqrt{3}\,\gamma_{m0}) + 0.9\, f_u\, A_{tn} / \gamma_{m1} \quad \ldots (5f)$$

Or

$$T_{db} = 0.9 f_u A_{vn} / (\sqrt{3}\,\gamma_{m1}) + f_y\, A_{tg} / \gamma_{m0} \quad \ldots (5g)$$

A_{vg} and A_{vn} = minimum gross and net area in shear along a line of transmitted force, respectively (along 1-2 in Fig. 6a or along A-B and D-C in Fig 6f), A_{tg} and A_{tn} = minimum gross and net area in tension from the hole to the toe of the angle, or next last row of bolts in plates perpendicular to the line of force, respectively (along 2-3 in Fig. 6a or along B-C in Fig. 6f).

It may be of interest to note that the American code has adopted the following block shear formula for angles, with a resistance factor of $\varphi = 0.75$.

$$\varphi T_n = \varphi\, [0.6\, f_y\, A_{vg} + 0.5\, f_u\, A_{tn}],$$

with $0.6\, f_u\, A^{vg} = 0.6\, fu\, A_{vn} \quad \ldots (5h)$

Appendix A gives a design example based on these provisions.

Design Of Compression Members

Compression members are prone to buckling and the buckling strength is influenced by various parameters such as shape of the cross-section, residual stress, initial crookedness and end restraint. Researchers in Australia, and European countries have realized that the effect of these variables may be taken into account by using multiple column curves[18, 19].

In the Indian code, the members subjected to axial compression are classified as per buckling curves a, b, c and d as given in Table 3. The multiple column curves in non-dimensional form are shown in Fig.7.

Table 3: Buckling Class Of Cross-Sections[1]

Cross Section	Limits	Buckling about axis	Buckling Curve
Rolled I Section	$h/b_f > 1.2$: $t_f \le 40$ mm	major	a
		minor	b
	40 mm < $t_f \le 100$ mm	major	b
		minor	c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	major	b
		minor	c
	t_f > 100 mm	major & minor	d
Welded I Section and rolled/ Welded I with cover Plates	t_f = 40 mm	major	b
		minor	c
	t_f > 40 mm	major	c
		minor	d
Welded Box Section	Generally (except as below)	any	b
	Thick welds and b/t_f < 30 mm	major	c
	h/t_w < 30 mm	minor	c
Round, Square or Rectangular tubes	Hot rolled	any	a
	Cold formed	any	b
Channel, Tee, Angle, and Solid sections		any	c
Built up member		any	c

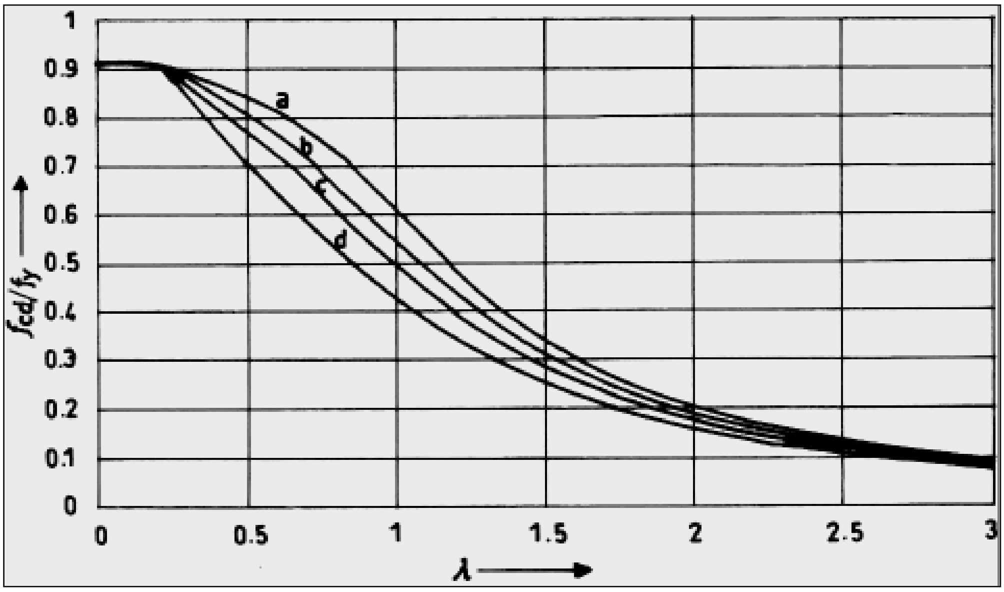


Fig. 7 Multiple Column Buckling Curves1

$\varphi = 0.5\ [1 + \alpha\ (\lambda - 0.2) + \lambda^2]$ . . . (6e)

α = imperfection factor that accounts for the effects of residual stresses and imperfections corresponding to different column curves as given in Table 4, λ = non-dimensional effective slenderness ratio = $\sqrt{(f_y/f_{cc})}$, f_{cc} = Euler buckling stress = $\pi^2E/(KL/r)^2$, KL/r = effective slenderness ratio, or ratio of effective length KL to appropriate radius of gyration, r, and γ_{m0} = partial safety factor for material strength.

Table 4: Imperfection factor, α				
Buckling curve	a	b	c	d
α	0.21	0.34	0.49	0.76

The design compressive strength, P_d of the member is given by

$P \leq P_d$ . . . (6a)

Where

$P_d = A_e\ f_{cd}$ . . . (6b)

A_e = effective sectional area (gross area minus holes not filled with rivets, bolts etc.)

f_{cd} = design compressive stress

$$= \frac{(f_y/\gamma_{m0})}{\phi + (\phi^2 - \lambda^2)^{0.5}} = \chi f_y/\gamma_{m0} \leq f_y/\gamma_{m0} \quad \ldots (6c)$$

Where χ = Stress reduction factor

$= 1/[\varphi + (\varphi^2 - \lambda^2)^{0.5}] \leq 1$ . . . (6d)

Table 5: Constants k_1, k_2, k_3				
No. of bolts at each end of connection	Gusset/Connecting Member fixity*	k_1	k_2	k_3
≥ 2	Fixed	0.20	0.35	20
	Hinged	0.70	0.60	5
1	Fixed	0.75	0.35	20
	Hinged	1.25	0.50	60
*Stiffness of in-plane rotational restraint provided by the gusset/ connecting member				

The code presents the stress reduction factor, χ and the design compressive stress, f_{cd}, for different buckling curves, yield stresses and effective slenderness ratios in tables, for the convenience of designers.

Design Of Angle Struts

Based on the research conducted at IIT Chennai, it is suggested that the flexural torsional buckling strength of single angles loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e as given below:

$$\lambda_e = \sqrt{[k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\varphi^2]} \quad \ldots (7)$$

where k_1, k_2, k_3 = constants depending upon end conditions as given in Table 5

$\lambda_{vv} = (L/r_{vv})\ /\ [\varepsilon\ \sqrt{\pi^2E/250}\,]$ . . . (7a)

$\lambda? = (b_1 + b_2)\ /\ [2\ \varepsilon t\ \sqrt{\pi^2E/250}\,]$ . . . (7b)

Where, L = Laterally unsupported length of the 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 leg, and ε = yield stress ratio = $(250/f_y)^{0.5}$

Design Of Beams (Flexural Members)

Short beams may attain its plastic moment capacity, provided a plastic or compact section is chosen. However, long beams are prone to lateral-torsional buckling, which results in reduced strength. It has been shown that a set of curves, similar to the multiple column curves can be generated for different cross-sectional shapes[19, 20]. The Indian code has adopted two curves for the design of laterally unsupported beams (for rolled and welded sections - by adopting two values of α_{LT}, as described below).

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

$$M \leq M_d \quad \text{. . . (8)}$$

Where M_d = design bending strength of the section.

The design bending strength of beam, adequately supported against lateral torsional buckling (laterally supported beam) is governed by the yield stress (section 8.2.1. of code). When a beam is not adequately supported against lateral buckling, the design bending strength is given by

$$M_d = \beta_b Z_p f_{bd} \quad \text{. . . (8b)}$$

Where

β_b = 1.0 for plastic and compact sections

= Z_e/Z_p for semi-compact sections

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

f_{bd} = design bending compressive stress, obtained as given below.

$$f_{bd} = \chi_{LT} f_y/\gamma_{m0} \quad \text{. . . (8c)}$$

Where

χ_{LT} = reduction factor to account for lateral torsional buckling given by:

$$\chi_{LT} = \frac{1}{\left[\phi_{LT} + \left(\phi_{LT}^2 - \lambda_{LT}^2\right)\right]^{0.5}} \leq 1.0 \quad \text{. . . (8d)}$$

in which $\varphi_{LT} = 0.5 [1 + \varphi_{LT} \{\lambda_{LT} - 0.2\} + \lambda_{LT}^2]$ . . . (8e)

The values of imperfection factor, α_{LT}, for lateral torsional bucking of beams is given by:

α_{LT} = 0.21 for rolled section and α_{LT} = 0.49 for welded section

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

$$\chi_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} = \sqrt{1.2 Z_e f_y / M_{cr}} \quad \text{. . . (8f)}$$

where, M_{cr} = elastic critical moment.

The code also gives a simplified equation for calculating M_{cr} for symmetric I sections. Annex E of the code provides methods for calculating M_{cr} for different beam sections, subject to different loadings and support conditions. Note the similarity of equation (8d) adopted for beams and equation (6d) adopted for columns.

Design Of Beam-Columns (Members Subject To Combined Forces)

Members subjected to combined bending and combined axial force should be checked for cross section strength and overall Member strength.

Table 6: Constants α_1, α_2		
Section	α1	α2
I and Channel	5n = 1	2
Circular tubes	2	2
Rectangular tubes	$1.66/(1-1.13n^2) \leq 6$	$1.66/(1-1.13n\alpha) \leq 6$
Solid rectangles	$1.73+1.8 n^3$	$1.73+1.8 n^3$
Note : $n = N/N_d$		

Check For Cross-Section strength

For plastic and compact sections the following interaction equation is suggested by the code.

$$\left[\frac{M_y}{M_{ndy}}\right]^{\alpha_1} + \left[\frac{M_z}{M_{ndz}}\right]^{\alpha_2} \leq 1.0 \quad \text{. . . (9)}$$

Conservatively, the following relationship may be used under combined axial force and bending moment.

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0 \quad \text{. . . (9a)}$$

Where, M_y, M_z = factored applied moments about the minor and major axis of the cross section respectively, M_{ndy}, M_{ndz} =design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone, (approximate expressions are given in the code for calculating these quantities), N = factored applied axial force (Tension T, or Compression F), N_d = design strength in tension = $A_g f_y / \lambda_{m0}$, M_{dy}, M_{dz} = design strength under corresponding moment acting alone (calculated as per Eqn. 8b), A_g = gross area of the cross section, and α_1, α_2 = constants as given in Table 6.

Check For Overall Member Strength

***Bending And Axial Tension*:** The reduced effective moment Meff under tension and bending should not exceed the bending strength due to lateral torsional buckling Md (Eqn. 8b). The reduced effective moment is given by,

$$M_{eff} = M - \psi T Z_e / A = M_d \quad \text{. . . (10)}$$

Where M and T are the factored applied moment and Tension, respectively, A is the area of cross-section and Z_e is the elastic section modulus of the section with respect to extreme compression fibre. The factor ψ is taken as 0.8 when

T and M vary independently; or otherwise taken as 1.0.

Bending And Axial Compression: The interaction equation for overall member buckling check is given by the code as

$$(P/P_{dy}) + (K_y\, C_{my} M_y/M_{dy}) + (K_{LT}\, M_z/M_{dz}) \le 1.0 \qquad \ldots (11a)$$

$$(P/P_{dz}) + (0.6 K_y C_{my} M_y/M_{dy}) + (K_z C_{mz} M_z/M_{dz}) < 1.0 \qquad \ldots (11b)$$

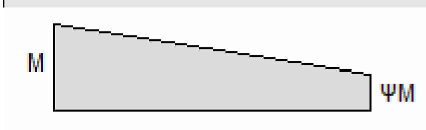
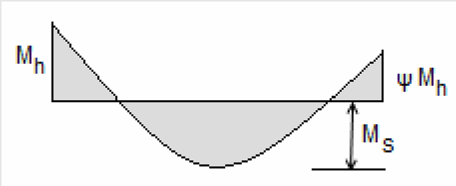
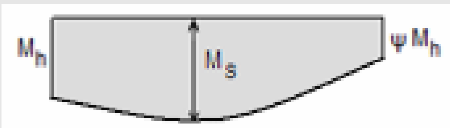
Where, C_{my}, C_{mz} = Equivalent uniform moment factor obtained from Table13.3, which depends on the shape of the bending moment diagram between lateral bracing points in the appropriate plane of bending, P = Factored applied axial compressive load, P_{dy}, P_{dz} = Design compressive strength under axial compression as governed by buckling about minor and major axis respectively (See Eqn. 6b), M_y, M_z = Maximum factored applied bending moments about minor and major axis of the member, respectively, M_{dy}, M_{dz} = Design bending strength about minor and major axis considering laterally unsupported length of the cross-section (see Eqn. 8b), and K_y, K_z, K_{LT} = Interaction factors as defined below.

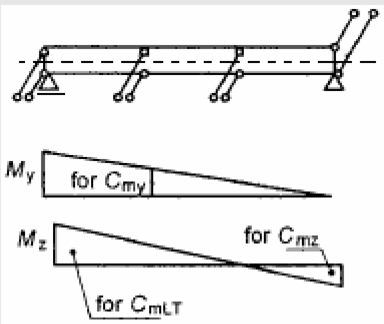
$$K_y = 1 + (\lambda_y - 0.2)_{ny} \le 1 + 0.8\, n_y \qquad \ldots (11c)$$

$$K_z = 1 + (\lambda_z - 0.2)_{nz} \le 1 + 0.8\, n_z \qquad \ldots (11d)$$

$$K_{LT} = 1 - \frac{0.1 \lambda_{LT} n_y}{(C_{mLT} - 0.25)} = 1 - \frac{0.1 n_y}{(C_{mLT} - 0.25)} \qquad \ldots (11e)$$

Table 7: Equivalent Uniform Moment Factor (Greiner & Lindner, 2006)

Bending moment diagram	Range		C_{my}, C_{mz}, C_{mLT}	
			Uniform Loading	Concentrated load
	$-1 \le \psi \le 1$		$0.6 + 0.4\,\psi \ge 0.4$	
	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0.2 + 0.8\,\alpha_s \ge 0.4$	$0.2 + 0.8\,\alpha_s \ge 0.4$
	$-1 \le \alpha_s \le 0$	$0 \le \psi \le 1$	$0.1 - 0.8\,\alpha_s \ge 0.4$	$-0.8\,\alpha_s \ge 0.4$
$\alpha_s = M_s / M_h$		$-1 \le \psi \le 0$	$0.1(1-\psi) - 0.8\alpha_s \ge 0.4$	$0.2(1-\psi) - 0.8\alpha_s \ge 0.4$
	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0.95 - 0.05\,\alpha_h$	$0.90 + 0.10\,\alpha_h$
	$-1 \le \alpha_h \le 0$	$0 \le \psi \le 1$	$0.95 + 0.05\,\alpha_h$	$0.90 + 0.10\,\alpha_h$
$\alpha_h = M_h / M_s$		$-1 \le \psi \le 0$	$0.95 + 0.05\alpha_h(1+2\,\psi)$	$0.90 + 0.05\alpha_h(1+2\,\psi)$
For members with sway buckling mode the equivalent uniform moment factor $C_{my} = C_{mz} = 0.90$				
C_{my}, C_{mz}, and C_{mLT} shall be obtained according to the bending moment diagram between the relevant braced points as below:				



Moment factor	Bending axis	Points braced in direction
C_{my}	z-z	y-y
C_{mz}	y-y	z-z
C_{mLT}	z-z	z-z

Where, n_y, n_z = Ratio of actual applied axial force to the design axial strength for buckling about minor and major axis respectively = (P/P_{dy}) or (P/P_{dz}), C_{mLT} = equivalent uniform moment factor for lateral-torsional buckling as per Table 7, which depends on the shape of the bending moment diagram between lateral bracing points, λ_y, λ_z = Non-dimensional slenderness ratio about the minor and major axis respectively, For example $\lambda_y = (f_y/f_{cr})^{0.5}$, where $f_{cr} = \pi^2 E/(KL/r)^2$, and λ_{LT} = non-dimensional slenderness ratio in lateral buckling = $(f_y/f_{cr,b})^{0.5}$ and $f_{cr,b}$ is the extreme fibre bending compressive stress corresponding to elastic lateral buckling moment which may be determined as per Table 14 of the code.

The above Indian Code provisions are based on the Eurocode 3 provisions and the improved interaction equations suggested by Greiner and Lindner (2006). They derived Eqns (11) after extensive statistical evaluations and calibration with available buckling results.

Note that the equations (11) as given in the code are complex for design office use, though they may be incorporated in a computer code. It may be of interest to note that the American code gives a simple equation for overall member strength[8].

Though the code gives some provisions for the earthquake resistant design, they are not comprehensive as those given in the American code[9]. After the Northridge (USA,1994) and Kobe (Japan, 1995) earthquakes, it was found that several column-base connections designed following previous design practices and guidelines did not perform satisfactorily. The damage to the base connections consisted mostly of excessive anchor rod elongation, unexpected early anchor rod failure, shear key failure, brittle base plate fracture, and concrete crushing (including grout crushing). However the code contains only provisions for base plates subjected to axial compression.

Summary & Conclusions

The last version of the Code of Practice for general construction in steel, IS 800:2007, was released in Feb. 2008. This article reviews the important features of the code. It may be noted that the present code is based on Limit States Method of design and hence is on par with the national codes of other countries. Several important topics, which were hitherto not included in the previous editions of the code, have been included. These are: methods of analysis (which include advanced analysis, using which we may eliminate the approximate and often confusing concept of effective length of members; moment-rotation relationships for semi-rigid connections are also given), fatigue (these provisions are important to structures such as bridges, cranes, and those supporting machinery), durability (durability has become one of the main factors for design due to the severe corrosion of several structures, especially in the coastal zones. The limited and dwindling steel ore resources and sustainability concepts also underline the importance of durability), fire resistance (the recent fire accidents in several multi-storey buildings and subsequent loss of life necessitates these clauses), and design against floor vibration. It includes the state-of-the-art knowledge available till now, which will result in rational design of steel structures, with acceptable margin of safety under several limit states.

Though some provisions are included for earthquake resistant design, they are not elaborate such as those available in other national codes. A few provisions (e.g. overall member strength check for members subjected to axial force and bending moments), though very accurate, are not suitable for normal design office - though they may be programmed for digital computer use (Equations are provided for the Woods Curves, in Annex D, which can be easily incorporated in computer programs). Though the code is expensive (Rs.1130), it does not provide any commentary such as those available in ACI or AISC codes, which will enable the users to understand the rationale of different clauses. A design aid similar to SP 16 is under progress and will be published by BIS soon. More information and examples based on the codal provisions may be found in Ref.3.

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Appendix A

The following example is given to explain the application of tension member design provisions of the code[3].

A tie member in a bracing system consists of two angles 75 x 75 x 6 bolted to 10 mm gusset, one on each side using single row of bolts (Fig.8a) and tack bolted. Determine the tensile capacity of the member and the number of bolts required to develop full capacity of the member. What will be the capacity if the angles are connected on the same side of the gusset plate and tack bolted (Fig.8b)? What is the effect on tensile strength if the members are not tack bolted?

Solution

a) Two angles connected to opposite side of the gusset as in Fig. 8a

(i) Design strength due to yielding of gross section

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

$A_g = 866\ mm^2$ (for a single angle)

$T_{dg} = 250 \times 2 \times (866/1.10) \times 10^{-3}$

$T_{dg} = 393.64$ kN

(ii) The design strength governed by tearing at net section

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

Assume single line of four numbers of 20mm diameter bolts (α=0.8)

$A_n = [(75 - 6/2 - 22)\ 6 + (75 - 6/2)\ 6]^2$

$A_n = (300 + 432)2 = 1464\ mm^2$

$T_{dn} =(0.8 \times 1464 \times 410/1.25)/1000 = 384.15$ kN

Therefore Tensile capacity = 384.15 kN

Design of bolts:

Choose edge distance = 35 mm

Capacity of bolt in double shear (Table 5.9 of Ref.3)

= 2 x 45.3 = 90.6 kN

Bearing capacity of the bolt does not govern as per Table 5.9 of Ref. 3

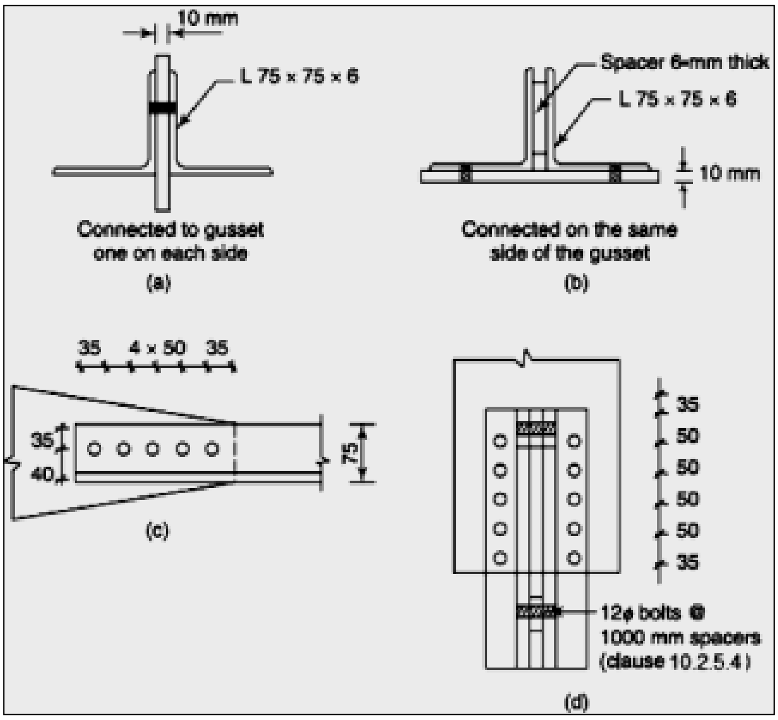

Fig.8 Example Problem

Hence strength of a single bolt = 90.6 kN

Provide 5 bolts

Total strength of the bolt = 5 x 90.6

= 453 kN > 384.15 kN

Hence safe.

Minimum spacing = 2.5 t = 2.5 x 20 = 50 mm

Provide a spacing of 50 mm

The arrangements of bolts are shown in Fig. 8c

Check for block shear strength: (clause 6.4)

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

$T_{db1} = [A_{vg}f_y / (v3_{m0}) + 0.9A_{tn}\ f_u / \lambda_{m1}]$

$T_{db2} = [0.9f_u\ A_{vn} / (v3\gamma_{m1}) + f_y\ A_{tg} / \gamma_{m0}]$

$A_{vg} = (4 \times 50 + 35)\ 6 = 1410\ mm^2$

$A_{vn} = (4 \times 50 + 35 - 4.5 \times 22)\ 6 = 816\ mm^2$

$A_{tn} = (35.0 - 22/2)6 = 144\ mm^2$

$A_{tg} = (35 \times 6) = 210\ mm^2$

$T_{db1} =$ [1410 x 250 / (v3 x 1.10) + 0.9 x 144 x 410 / 1.25] x 10^{-3} = 227.5 kN

$T_{db2} =$ [0.9 x 410 x 816/ (v3 x 1.25)] + 250 x 210 / 1.10] x 10^{-3} = 186.8 kN

For double angle block shear strength = 2 x186.8

= 373.6 kN

Therefore Tensile capacity = 373.6 kN (smallest of 393.64 kN, 384.15 kN and 373.6 kN)

b) Two angles connected to the same side of the gusset plate (Fig. 8b)

i. Design strength due to yielding of Gross section = 393.64 kN

ii. Design strength governed by tearing at net section = 384.15 kN

Assuming 10 bolts of 20 mm diameter, five bolts in each connected leg

Capacity of M20 bolt in single shear = 45.3 kN

Total strength of bolts = 10 x 45.3 = 453 kN > 393.64 kN

Hence the connection is safe.

The arrangement of bolts is shown in Fig. 8d. Since it is similar to the arrangement in Fig. 8c, the block shear strength well be same, i.e. 373.6 kN.

Hence the tensile capacity = 373.6 kN

The tensile capacities of both the arrangements (angles connected on the same side and connected to the opposite side of gusset) are same as per the code though the load application is eccentric in this case. Moreover, the number of bolts is 10 whereas in case (a) we used only 5 bolts since the bolts were in double shear.

c) If the angles are not tack bolted, they behave as single angles connected to gusset plate.

In this case also the tensile capacity will be the same and we have to use 10 numbers of M20 bolts. This fact is confirmed by the test and FEM results of Usha, 2003, who states that "the net section strength of double angles on opposite sides of the gusset and tack connected adequately over the length is nearly the same as that of two single angles acting individually. Current design provisions indicating greater efficiency of such double angles are not supported by test and FEM results".