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State-of-the-art review of seismic design of steel moment resisting frames – Part I : General considerations and stability provisions

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This paper presents a state-of-the-art review of seismic design provisions for steel moment resisting frames given in American and Indian codes. Unlike reinforced concrete structures, steel structures are usually made of plate-like components, and their stability under compressive stresses is vital. The merits of the different design methods and the importance of the different levels of stability provisions in the codes are discussed. Eventhough earthquake-resistant design of steel structures is not formally addressed in Indian codes, this paper draws a parallel of the available Indian codal provisions with those in the American codes, with a view to identify the level of earthquake shaking, to which the Indian codal provisions may be applicable.

Steel structures have, in general been able to withstand severe earthquake shaking, owing to their intrinsic ductility. Steel with its large strength-to-weight ratio is considered as a good earthquake-resistant material. Nearly equal response of steel, under both tensile and compressive load, enhances its performance under cyclic loading. However, a major consideration in the seismic design of steel structures is the stability limit state. As the structural steel members are generally made up of plate-like elements, stability of these elements is essential to achieve a good hysteretic performance of the whole structure. Under severe earthquake shaking during past earthquakes, most of the failures are seen to be due to local buckling of elements having large width-to-thickness ratio, flexural buckling of long columns, and lateral-torsional buckling of beams and beam-columns. In addition, failure of the connections and fracture of welds subjected to stress concentration, fracture of plates owing to large strains caused by local or flexural buckling, tearing of welded connection between beam and column flange due to large local deformations, P - Δ effect, and low-cycle fatigue under cyclic loading, are responsible for not having the desired performance in steel structures.

The main aim of earthqquake-resistant design of steel structures is to achieve a stable post yield behaviour of the structure, irrespective of the design method. Clearly, the post-yield behaviour is significantly influenced by the design method, and hence, the design methods which explicitly recognise and account for the post-yield response, are preferred. However, the effectiveness of elastic design method may be enhanced by introducing appropriate stability provisions related to post-yield seismic behaviour and by improving the detailing of connections.

CHOICE OF STRUCTURAL SYSTEMS

The common typologies of seismic-resistant steel structures are: (a) Moment Resisting Frames (MRF), (b) Concentrically Braced Frames (CBF), (c) Eccentrically Braced Frames (EBF), and (d) Truss Moment Resisting Frames (TMF). Also special ductile provisions are available to build Special Moment Resisting Frames (SMRF), Special Concentrically Braced Frames (SCBF), and Special Truss Moment Resisting Frames (STMF). The response reduction factors in Table 1, give an idea of the relative overstrength and ductility that can be provided by each of these structural systems under seismic loading. In India, MRFs and CBFs are used, but special ductility provisions to make them earthquake-resistant are not available in Indian codes. A brief discussion[1] on strength, rigidity, ductility and energy

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dissipating capacity of each of these structural systems is given below:

TABLE I RESPONSE REDUCTION FACTORS R[5]	
LATERAL FORCE RESISTING SYSTEM	R
Steel special moment resisting frame	8.5
Steel ordinary moment resisting frame	4.5
Steel eccentrically braced frame	7.0
Steel ordinary braced frame	5.6
Steel special concentrically braced frame	6.4
Steel eccentrically braced frame with special moment resisting frame	8.5
Steel eccentrically braced frame with ordinary moment resisting frame	4.2
Steel ordinary braced frame with special moment resisting frame	6.5
Steel ordinary braced frame with ordinary moment resisting frame	4.2
Steel special concentrically braced frames with special moment resisting frame	7.5
Special truss moment resisting frame	6.5

Two districnt collapse mechanisms are possible in MRF (Fig.1). The maximum energy dissipation capacity is associated with the beam-hinge mechanisms, in which the energy dissipation zones are primarily at the end of the beams (Fig.1a). Hence a large number of energy dissipation zones are formed. This is in contrast with the column-hinge mechanism (Fig.1b), wherein only a few energy dissipation zones are required to collapse the frame. The design approach which leads to the beam-hinge mechanism, known as the strong-column weak-beam approach, is therefore preferred. MRFs provide satisfactory strength and possess excellent ductility, but tend to behave too flexibly,

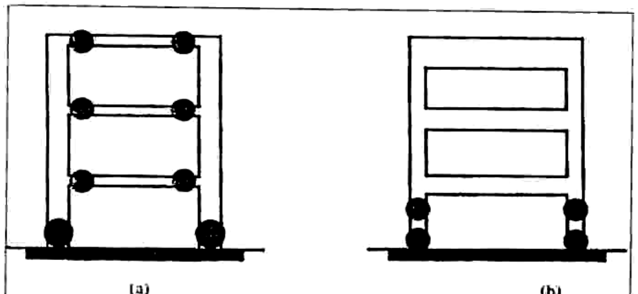

FIG.1 TWO POSSIBLE COLLAPSE MECHANISMS IN MRFs:
(a) BEAM-HINGE MECHANISM,
(b) COLUMN-HINGE MECHANISM

especially in medium to high-rise buildings. Hence, in medium to high-rise MRF buildings, the design is usually governed by the drift criteria rather than the strength criteria. In CBFs (Fig.2), the energy dissipation zones are possible mainly in the diagonal brace members, which undergo cyclic yielding under fluctuating axial forces. Under axial compressive forces, which is subjected to alternate compressive and tensile forces, shows unsatisfactory in-

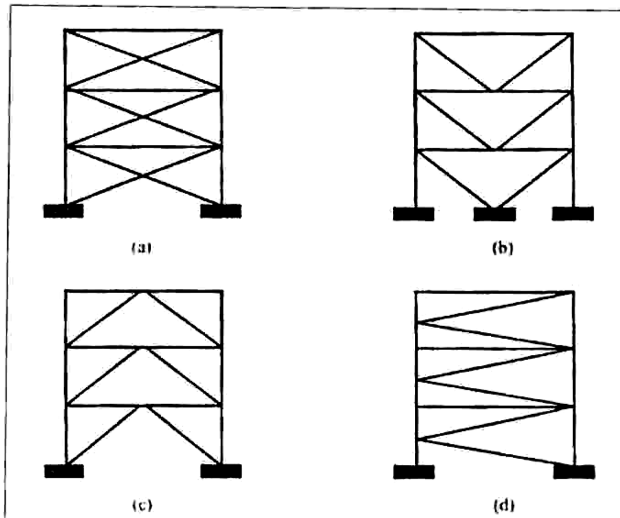

FIG.2 CONCENTRICALLY BRACED FRAMES
(a) X-BRACED FRAME (b) V-BRACED FRAME
(c) INVERTED V-BRACED FRAME, (d) K-BRACED FRAME

elastic performance (Fig.3). Also, the energy dissipation capacity of the system degrades as the number of cycles increase. Experimental studies reveal that V-braced frames suffer both from strenth degradation and stiffness deterioration, while X-braced frame frames undergo only stiffness deterioration.

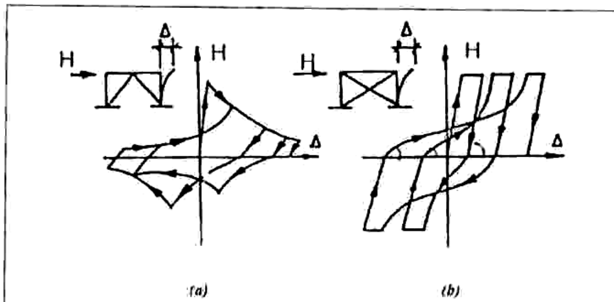

FIG.3 CYCLIC BEHAVIOUR OF CONCENTRICALLY BRACED FRAME (a) V-BRACED FRAME, (b) X-BRACED FRAME

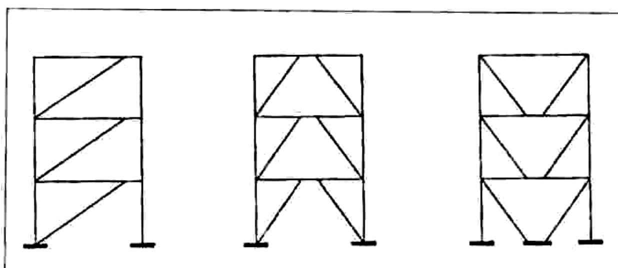

FIG.4 ECCENTRICALLY BRACED FRAMES (a) D-BRACED FRAME (b) K-BRACED FRAME, (c) V-BRACED FRAME

In EBFs, the energy input by the earthquake is dissipated by the inelastic shearing and bending of the links, i.e., portions of the beam between the eccentric locations of the diagonal braces in the frame (Fig.4). The EBFs are designed such that, plastic hinges which dissipate energy are located only in the links. All other structural components are designed to remain elastic under the maximum forces that can occur in the structure.

Both MRFs and EBFs have a large number of energy dissipation zones, and hence have good ductility and energy dissipation capacity. Therefore, these are considered superior to CBFs for use in seismically active zones. However, the lateral stiffness of MRFs is less than that of CBFs and EBFs.

Typically, in the case of long span beams, eg., in bridges, TMFs are commonly used (Fig.5a). In American practice, STMFs are designed with a ductile segment consisting of a combination of top and bottom chords, with or without X bracing. A deformed shape of this frame (Fig.5b) shows the post-yield performance of this ductile segment. The plastic hinges are likely to form in these ductile members and thus absorb a lot of earthquake energy without damaging the rest of the structure.

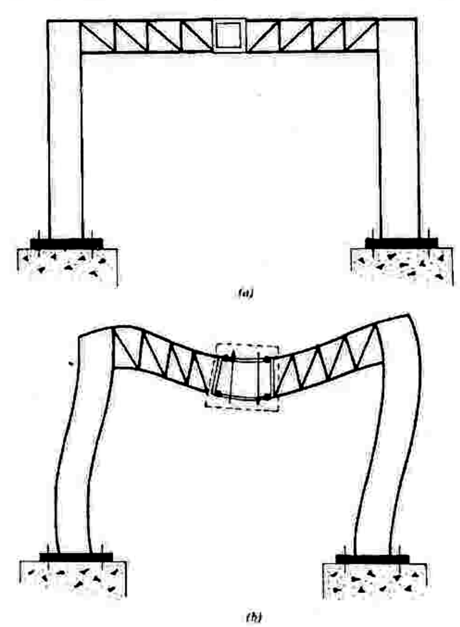

FIG.5 A SPECIAL TRUSS MOMENT FRAME (a) UNDEFORMED CONFIGURATION SPECIAL DUCTILE ELEMENTS, (b) DEFORMED CONFIGURATION SHOWING THE PLASTIC MOMENT HINGES IN THE SPECIAL DUCTILE ELEMENTS

DESIGN METHODS

Three design methods are adopted in the seismic design of steel structures. These methods are: (a) Allowable Stress Desin (ASD)[2] Method, (b) Plastic Design (PD)[2] Method and (c) Load and Resistance Factor Design (LRFD)[3] Method. The ASD method is the oldest one, wherein the design strength calculated using permissible stresses are ensured to be larger than the applied loads. The PD approach uses the full strength of the material, but scales up the service loads by load factors. The LRFD method combines the above two methods; the ideal strength of the structure is reduced and the service loads are enhanced.

The present American codes permit seismic design by any of the above three methods. However, certain provisions are not available in the ASD method, e.g., the joint panel zone design provisions which indirectly accounts for the inelastic behaviour. However, the American codes have calibrated the three design methods in such a way that stability provisions are covered.

In India, the ASD is popular, even though the PD method is also followed; the LRFD approach is yet to come into the Indian design practice. Suitability of the three design methods, from the stand point of seismic design of steel structures, is discussed in the following sections with respect to American and Indian codes.

SEISMIC DESIGN CODES

American Codes

Presently, three organisations publish design specifications in USA for seismic design of steel structures. The American Institute of Steel Construction, publishes: (a) Allowable Stress Design (AISC-ASD)[2] and the plastic Design (AISC-PD)[2], (b) Metric Load and Resistant Factor Design (AISC-LRFD)[3], and (c) Seismic Provisions for Structural Steel Buildings (SPSSB 97)[4]. The International Conference of Building Officials publishes the Uniform Building Code (UBC 97)[5] and the Federal Emergency Management Agency, publishes model code provisions under the National Earthquake Hazard Reduction Program (NEHRP 97)[6].

The AISC ASD and the PD specifications were last revised in 1989 by re-organising the provisions to be consistent with the older version (1986) of the LRFD specifications. The 1986 version of AISC-LRFD was revised in 1993 and in 1994, it was converted to metric specifications and given the name Metric Load and Resistance Factor Design. However, none of the above specifications are sufficient for seismic design of steel structures. The SPSSB specifications[4] provide additional requirements for steel structures in high seismic zones.

UBC 97 and NEHRP 97 specifications refer to the ASD, the PD and the LRFD approaches of AISC. These, further refer to the AISC-SPSSB[4] specifications for additional provisions for seismic design of steel structures. UBC 97 has clear guidelines for drift calculations. NEHRP 97 has additional seismic provisions related to steel deck diaphragms, cables, etc.

AISC-ASD and AISC-PD Methods: The ASD method wherein the focus is on service load conditions while satisfying the safety requirements of the structure. In the ASD approach, the reserve strength of material beyond the elastic limit is not considered. Structures that are expected to suffer low strain levels and remain within the elastic limit, i.e., when the loads and a structure are predominantly of dead load, with small or negligible live, wind, or earthquake loads, the ASD approach is sufficient for designing the structure. However, if the structure is expected to sustain large earthquake loads, the ASD approach is considered somewhat deficient. This is because the structure is designed only for a small fraction of maximum expected seismic loads, and the stable post-yield behaviour the structure is relied upon, to account for the remaining forces.

In the PD method, the real strength of members and structural system is estimated, and checked against service loads multiplied by a load factor. This load factor compares well with the product of the, factor of safety employed in the ASD method and the shape factor. Thus, the reserve strength up to yield point is considered in PD approach. However, this design method does not account for the large post-yield strains developed under severe earthquake shaking, the residual stress and the strain hardening effects. Stability provisions specified in PD approach, prevent the instabilities like local buckling, lateral-torsional buckling, and flexural buckling for low to medium range of post-yield strains only. Hence, the PD approach seems to be an appropriate design method for designing structures in low to moderate seismic zones.

AISC - LRFD Specifications: The AISC-LRFD specifications identify both strength (yielding) and stability (buckling) limit states. Depending upon the degree of uncertainty associated with the type of loads, the strength design philosophy uses factored service loads. These factored loads are compared with the nominal strengths obtained by multiplying the yield strength with suitable strength reduction factors (Table 2). The LRFD provisions are usefule for earthquake resistant structures, as they consider various instabilities and assure that high inelastic strains can be sustained.

TABLE 2
STRENGTH REDUCTION FACTORS

NATURE OF FORCE		REDUCTION FACTORS
Tension:	Yielding Rupture	0.90 0.75
Compression:	Buckling	0.85
Flexure:	Yielding Rupture	0.90 0.75
Shear:	Yielding Rupture	0.90 0.75
Torsion:	Yielding Rupture	0.90 0.90
Welds		0.74 to 0.90
Bolts		0.75 to 1.00
Connecting Elements		0.60 to 0.90
Flanges and Webs with concentrated forces		0.75 to 1.00

AISC-SPSSB Code: This code specifies the special seismic requirements for ordinary moment frames (OMF), special moment frames (SMF), intermediate moment frames (IMF), ordinary concentrically braced frames (OCBF), special truss moment frames (STMF), special concentrically braced frames (SCBF) and eccentrically braced frames (EBF), in conjunction with other AISC specifications. Applicability of the SPSSB provisons again depends on building category for which it refers to the relevant building codes, e.g., UBC 97 and NEHRP 97. In particular these specifications are stated to be applicable for the most severe seismic conditions, e.g., seismic design category D (or higher) of NEHRP.

Indian Codes

The specifications for the design of different structural steel elements, like beams, columns and connections, by elastic and plastic design methods, are given in IS:800-1984[7]. The plastic theory for the design of steel structures, is described in the Indian Standard Handbook SP:6 (Part 6)-1972[8]. However, the plastic design method in the handbook focuses primarily on gravity and wind forces only without any specific reference to seismic design.

The design practice of steel structures in India is based on the allowable stress design approach (IS-ASD). Eventhough the plastic design approach (IS-PD) guidelines are outlined in standard code[7] and handbook[8], these are still not popular.

STABILITY ISSUES

In earthquake resistant design of steel structures, the stability of the structural framework, consisting of beams, columns and the plate elements, is extremely important. The stability of a steel structure depends on the level of strain and hence, the whole issue of the stability is discussed with strain (epsilon) as the governing parameter.

Major instabilities in steel structures are in the form of flexural buckling of columns, local buckling of beam-column flanges and webs, and lateral-torsional buckling of beams. The main reason for the above instabilities under earthquake loading is the unusually high strain demand (several times the yield strain). With reference to the idealised stress-strain diagram of mild steel (Fig.ure 6), four basic strain states can be defined: (a) sub-yield strain range, $\varepsilon \leq 0.5\,\varepsilon_y$, (b) inelastic strain range,

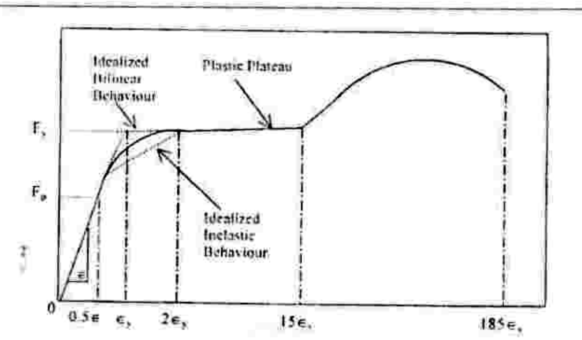

FIG.6 IDEALIZED STRESS–STRAIN DIAGRAM OF MILD STEEL[9]
(Figure Not To Scale)

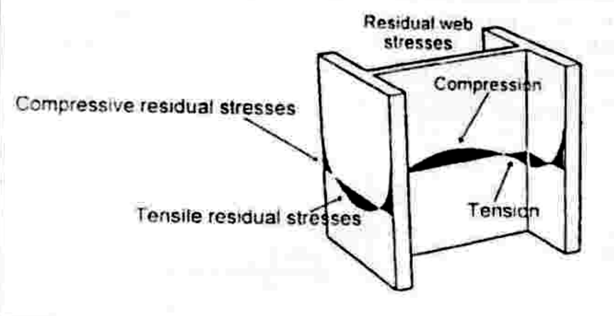

FIG.7 RESIDUAL STRESSES IN ROLLED SECTIONS[9]

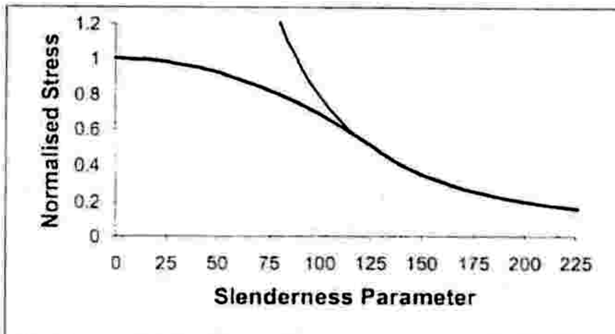

FIG.8 SSRC BASIC STRENGTH CURVE FOR COLUMN[9]

$0.5\varepsilon_y < \varepsilon \leq 2\varepsilon_y$ (c) perfectly plastic strain range, $2\varepsilon_y < \varepsilon \leq \varepsilon_{st}$ (d) strain-hardened strain range, $\varepsilon \geq \varepsilon_{st}$. The above deviates from the usual understanding of the yield strain ε_y as the proportional limit, in the sense that for structural steel inelasticity begins at a strain lower than ε_y, due to the presence of residual stress (Fig.7). Depending upon the capability of sustaining different strain levels, structural sections are classified as compact sections, non-compact sections and slender sections. A compact section shows a stable behaviour at low to medium levels of strain in the perfectly plastic strain range ($2\varepsilon_y - 6\varepsilon_y$), thus ensuring that the section can reach its full strength without any buckling. However, under severe earthquake shaking, the strain demand may be as much as , or even higher in the strain hardening range and in that case it is not sufficient to just achieve the full moment capacity; rather, the section should be able to sustain higher post-yield strains. This is achieved by controlling the compactness criteria, like width-to-thickness ratio of flanges and depth-to-thickness ratio of webs. A non-compact section can develop a small amount of post-yield inelastic strain, but does not develop the full moment capacity. The slender sections become unstable even at sub-yield levels of strain and are clearly unsuitable for structures, which are required to resist moderate to strong earthquake shakin.g

Stability of Columns

Stability of axially-loaded column is governed by its slenderness ratio. The AISC-ASD approach permits a maximum slenderness ratio of 200, while IS-ASD allows a maximum value of 250. On the other hand. The AISC-PD and IS-PD methods limit the maximum slenderness ratio to 126 and 120, respectively, to avoid elastic buckling of column. This can be considered as a useful ductile provision for structures in low seismic zones.

In high seismic zones wherein strain demand is much higher, a limit on slenderness ratio of 35[10] can help the section to reach almost its full strength (Fig. 8). For columns to remain stable in the strain-hardening range, the slenderness ratio should be as low as 16[9], which is almost impractical to adopt. Hence, the American design practice follows a slenderness ratio between 50 and 60[9]. Incidentally, such a straingent requirement is also available in IS-PD method, wherein the maximum slenderness ratio of all columns in the frame are limited to 50.

Stability of Beam-Column Component Plates

The column and beam sections, where hot-rolled or built-up, consist of plate elements, which should be so proportioned that the stability limit state of each of them is beyond the strength limit state, at which the full capacity of the cross-section is reached. For Indian standard hot-rolled sections the AISC requirements of unsupported width-to-thichness ratio based on compact, non-compact and seismic criteria, are given in Table 3. It shows that most of the Indian standard sections are free from local buckling of web. However, a few of the wide flanged sections fail to satisfy the unsupported width-to-thickness requirement for flange. In plate girders, where the designer can select the width-to-thickness ratio of the plates, care must be taken to satisfy these stability requirements. The stability criterion is discussed below from the view points of (a) local buckling of the flanges and web, and (b) lateral-torsional buckling of the member.

Flange and Web Local Buckling: From the classical elastic buckling analysis of a plate, the critical stress F_{cr} is given by:

$$F_{cr} + \frac{K}{(b/t)^2}\left[\frac{\pi^2 E}{12(1-\mu^2)}\right] \tag{1}$$

where K reflects boundary condition of unloaded edges (Table 4), E is modulus of elasticity of steel, b is the unsupported width of plate, t is the thickness of the plate, and μ is Poison's ratio. Due to residual stresses and other initial imperfections, inelastic buckling begins at a stress value of $F_{cr} = F_y/2$ (Fig.9) and the b/t ratio required for this stress level is $38K$. This considerably high for beam and column flanges, and for beam and column webs this is even much higher due to larger K values (Table 4). higher values of K are chosen for webs because they have better end support conditions between top and bottom flanges. Thus, plate stability is seldom a consideration, when beams or columns are designed to remain elastic within the proportional limit.

TABLE 3
CATEGORISATION OF INDIAN ROLLED SECTIONS BASED ON FLANGE WIDTH-TO-THICKNESS AND WEB DEPTH-TO-THICKNESS RATIO AS PER LRFD SPECIFICATION[3] FOR COMPACT AND NONCOMPACT CLASSIFICATION AND AS PER SPSSB[4] SPECIFICATION FOR SEISMIC CLASSIFICATION

Section	Unsupported width-to-thickness ratio		Satisfaction of compact, Non-Compact and Seismic Criteria			
	Flange	Web	Compact/ Non-Compact	Seismic	Compact/ Non-Compact	Seismic
ISMB 450	4.3	44.2	Compact	Yes	*	*
ISMB 500	5.3	45.6	Compact	Yes	*	*
ISMB 550	5.0	45.7	Compact	Yes	*	*
ISMB 600	5.0	46.5	Compact	Yes	*	*
ISWB 250	11.1	34.6	Non-Compact	No	Compact	Yes
ISWB 300	10.0	37.8	Compact	No	Compact	Yes
ISWB 350	8.7	40.9	Compact	No	Compact	Yes
ISWB 400	7.5	43.5	Compact	Yes	*	*
ISWB 450	6.5	45.6	Compact	Yes	*	*
ISWB 500	8.5	47.5	Compact	Yes	*	*
ISWB 550	7.1	49.0	Compact	Yes	*	*
ISWB 600	5.8	49.8	Compact	Yes	*	*
ISHB 200	11.1	23.3	Non-Compact	No	Compact	Yes
ISHB 200	11.1	29.8	Non-Compact	No	Compact	Yes
ISHB 250	12.9	26.2	Non-Compact	No	Compact	Yes
ISHB 250	12.9	33.4	Non-Compact	No	Compact	Yes
ISHB 300	11.9	29.7	Non-Compact	No	Compact	Yes
ISHB 300	11.9	36.7	Non-Compact	No	Compact	Yes
ISHB 350	10.8	32.4	Compact	No	Compact	Yes
ISHB 350	10.8	39.4	Compact	No	Compact	Yes
ISHB 400	9.8	35.3	Compact	No	Compact	Yes
ISHB 400	9.8	41.2	Compact	No	Compact	Yes
ISHB 450	9.1	37.4	Compact	No	Compact	Yes
ISHB 450	9.1	43.0	Compact	No	*	*
ISLC 125	10.0	25.4	Compact	No	Compact	Yes
ISLC 150	9.6	28.0	Compact	No	Compact	Yes
ISLC 225	8.8	35.3	Compact	No	Compact	Yes
ISLC 250	9.4	37.5	Compact	No	Compact	Yes
ISLC 350	8.0	43.9	Compact	Yes	*	*
ISLC 400	7.1	46.5	Compact	Yes	*	*

* Note : For webs, the seismic requirement of the depth to thickness ratio varies betwen 43-70 depending on the axial load.

TABLE 4
VALUES OF BOUNDARY CONDITION FACTOR K IN PLATE BUCKLING[10]

Condition Condition	K	Component
Fixed – Fixed	6.97	Web of column
Fixed – Simply supported	5.42	
Simply supported – Simply supported	4.0	Web of beam
Fixed – Free	1.28	Column flange
Simply supported – Free	0.43	Beam Flange

The critical stress in Eq.(1) can also be written in terms of the normalised slenderness parameter λ as:

$$\frac{1}{\lambda^2} = \frac{F_{cr}}{F_y} \tag{2}$$

and using Eq.(1):

$$\lambda^2 = \left(\frac{b}{t}\right)^2 \frac{F_y}{K} \left[\frac{2(1-\mu^2)}{\pi^2 E}\right] \tag{3}$$

This expression is not valid when critical stress exceeds the proportional limit, $F_p = F_y / 2$, because residual stresses if present, may cause yielding of a part of the cross-section even at lower strain level than the yield strain. Accordingly, the following normalised transition curve, also called the inelastic curve, that joins the point (F_p, λ_e) on the elastic curve at the proportional limit to the point (F_p, λ_0) at which strain hardening is expected, is recommonded[9].

$$\frac{F_{cr}}{F_y} = 1 \left(1 - \frac{F_r}{F_y}\right)\left(\frac{\lambda - \lambda_0}{\lambda_e - \lambda_0}\right) \tag{4}$$

where λ_e and λ_0 are the slenderness parameters corresponding to the starting of the elastic and strain hardening states, respectively; and F_r is the residual stress. Experiments have shown that λ_e is around 1.41 and λ_0 = 0.46 for component plates[9]. The transition curve drawn for plate element using Eq.(4) is shown in Figure 9. The limiting width-to-thickness ratios of flange and web plates of I-shaped sections, including hybrid sections and channels, as specified in the American and Indian codes are discussed in the following sub-sections.

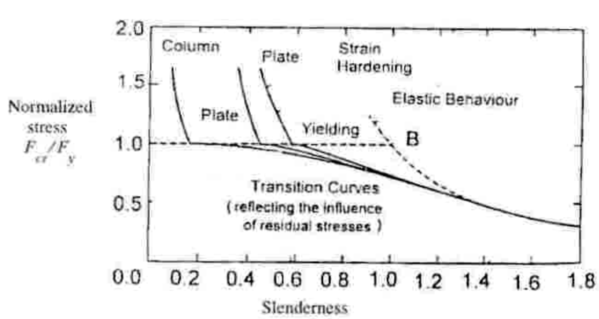

FIG.9 NORMALIZED CRITICAL STRESS FOR PLATE ELEMENTS AS FUNCTION OF SLENDERNESS PARAMETERS λ[9]

AISC-ASD and IS-ASD Provisions: The AISC-ASD specification limits the unsupported width-to-thickness ratio for flanges of I, T and channel sections to $171/\sqrt{F_y}$ and $250/\sqrt{F_y}$ for compact and non-compact sections, respectively. These values[9] have been based on $\lambda_y = \lambda = 0.46$. A value of $K = 0.76$ reflecting the end conditions between those of fixed-fixed and both ends simply supported, gives $b/t = 171/\sqrt{F_y}$. If b/t ratio exceeds $171/\sqrt{F_y}$, the AISC-ASD approach considers the section to be non-compact, even though it is still capable of attaining yield stress in the extreme fibre. For $F_{cr} = F_y$ ($\lambda = 1$) on the elastic curve, the b/t ratio from Eq.(3) is 426.5 and the inelastic curve gives $F_{cr} = 0.72F_y$. The above b/t ratio of 426.5 $426.5\sqrt{K/F_y}$ is proportionately reduced to get 307 $\sqrt{K/F_y}$. Choosing a value of $K = 0.66$, this ratio becomes $250\sqrt{F_y}$.

In case of webs with axial loads, AISC-ASD specifications limit the web depth-to-thickness ratio for compact sections as:

$$\left(\frac{b}{t}\right) \le \begin{cases} \dfrac{1686}{\sqrt{F_y}}\left(1 - 3.74\dfrac{f_a}{F_y}\right) & \text{for } \dfrac{f_a}{F_y} \le 0.16 \\ \dfrac{677}{\sqrt{F_y}} & \text{for } \dfrac{f_a}{F_y} > 0.16 \end{cases} \tag{5}$$

Here, f_a is the axial stress under design load. The same for non-compact sections is given by $2000/\sqrt{F_y}$.

The IS-ASD approach limits the maximum unsupported length of flange-to-thickness as $256/\sqrt{F_y}$, which is comparable to the AISC-ASD value of $250/\sqrt{F_y}$ for non-compact section. The web depth-to-thickness ratio is limited to 85 in absence of axial load, which is conservative in comparison to 107 (obtained from Eq.(5), for f_a equal to zero) given by AISC-ASD specifications.

AISC-PD and IS-PD Provisions: In the plastic design method the full strength of member has to be developed and hence all the provisions are based on compact sections. The AISC-PD method restricts the maximum width-to-thickness ratio of beam flange to $137/\sqrt{F_y}$, which is almost the same as the value of 8.5 specified by IS-PD method for $F_y = 250/\text{mm}^2$. For webs with axial load P, the AISC-PD specification gives the limiting web depth-to-thickness ratio as:

$$\left(\frac{b}{t}\right) \le \begin{cases} \dfrac{1085}{\sqrt{F_y}}\left(1 - 1.4\dfrac{P}{P_y}\right) & \text{for } \dfrac{P}{P_y} \le 0.27 \\ \dfrac{677}{\sqrt{F_y}} & \text{for } \dfrac{P}{P_y} > 0.27 \end{cases} \tag{6}$$

and the IS-PD specifications give

$$\left(\frac{b}{t}\right) \le \begin{cases} \dfrac{1120}{\sqrt{F_y}}\left(1 - 1.43\dfrac{P}{P_y}\right) & \text{for } \dfrac{P}{P_y} \le 0.27 \\ \dfrac{688}{\sqrt{F_y}} & \text{for } \dfrac{P}{P_y} > 0.27 \end{cases} \tag{7}$$

Eqs. (6) and (7) vary within 3% of each other.

AISC-LRFD Provisions: The AISC-LRFD provisions recommend a maximum width-to-thickness ratio of $171/\sqrt{F_y}$ and $355/\sqrt{F_y}$ for flanges of compact and non-compact sections, respectively. Under seismic conditions, a value of 137 And is recommended. Further, for axially-loaded webs of compact sections, AISC-LRFD provision requires that:

$$\left(\frac{b}{t}\right) \le \begin{cases} \dfrac{1680}{\sqrt{F_y}}\left(1 - \dfrac{2.75\,P_u}{\phi_c P_y}\right) & \text{for } \dfrac{P}{\phi_c P_y} \le 0.125 \\ \dfrac{500}{\sqrt{F_y}}\left(2.33 - \dfrac{P_u}{\phi_c P_y}\right) \ge \dfrac{667}{\sqrt{F_y}} & \text{for } \dfrac{P}{\phi_c P_y} > 0.125 \end{cases} \tag{8a}$$

where P_u is the factored axial load and ϕ_c is the strength reduction factor for column axial load design.

The same for non-compact section is given by:

$$\left(\frac{b}{t}\right) \le \frac{2550}{\sqrt{F_y}}\left(1 - \frac{0.74P}{\phi_c P_y}\right) \tag{8b}$$

AISC-SPSSB Provisions: SPSSB specifications for the maximum width-to-thickness ratio for flange is given by $137/\sqrt{F_y}$ and that for web under axial load is given by:

$$\left(\frac{b}{t}\right) \le \begin{cases} \frac{1370}{\sqrt{F_y}}\left(1 - \frac{1.54P_u}{\phi_c P_y}\right) & \text{for } \frac{P}{\phi_c P_y} \le 0.125 \\ \frac{500}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\phi_c P_y}\right) \ge \frac{667}{\sqrt{F_y}} & \text{for } \frac{f_a}{F_y} > 0.125 \end{cases} \tag{9}$$

Lateral-Torsional Buckling of Compression Flange

When a beam is bent about its major axis, it may twist excessively before reaching the strength limit state (lateral-torsional buckling), due to the flexurally induced axial stresses in the compression flange. However, the design of these bracing systems to provide the required torsional stability under elastic and post-yield conditions, is a major consideration. A compact section can develop full plastic moment, only if its laterally unbraced length is sufficiently small to avoid lateral-torsional buckling. From elastic flexural stability theory, the critical moment M_{cr} is given by:

$$M_{cr} = \sqrt{\pi^2 EI_y GJ/L^2 + \pi^4 E_2 C_w I_y/L^4} \tag{10}$$

where EI_y is the weaker axis flexural rigidity, GJ is the torsional rigidity, C_w is the warping constant, and L is the unbraced length of the beam. This expression is derived for the case of uniform moment throughout the length of the beam and the top flange of the beam is free to translate laterally. The first term accounts for Saint Venant's torsion and the second term for warping torsion. The expression is modified with a multiplying factor C_b to account for moment gradient as;

$$M_{cr} = C_b \frac{\pi}{L}\sqrt{EI_y \; GJ + \left(\frac{\pi E}{L}\right)^2 I_y \; C_w} \tag{11}$$

The following discussions of the code provisions use Eq.(11) as the basis. All the following expressions for maximum unbraced length of compression flange, to avoid lateral-torsional buckling, are for compact sections. These do not consider the interaction between local and lateral-torsional buckling, and hence no provisons for maximum unbraced lengths of compression flange are specified for non-compact sections. Further, these specifications are for only I-shaped sections including hybrid sections and channels.

AISC-ASD and IS-ASD Provisions: The AISC-ASD specifications for the maximum ubraced length L_c is:

$$L_c = Min\left[\frac{138888}{\left(d_w/A_f\right)F_y} \; ; \; \frac{200b_f}{\sqrt{F_y}}\right] \tag{12}$$

where A_f is the flange area, d_w is depth of the web and b_f is the flange width. The first limit is arrived at considering the Saint Venant's torsion term in Eq.(11).

For sections made up of thick plates, only Saint Venant's torsional stiffness will dominate. For such sections, the weaker axis radius of gyration r_y is approximated as $0.2b_f$ and the torsional constant J is approximated[9] as $0.28\, At_f^2$. Using these approximations and $C_b = 1.0$, in the first term of Eq.(11), the critical moment M_{cr} is obtained as:

$$M_{cr} = 0.23EAA_f/L_c \tag{13}$$

where, A is the total cross-sectional area of the section and t_f is the flange thickness. The critical moment M_{cr} can also be obtained as:

$$M_{cr} = F_{cr} S \approx 0.34\, Ad_w \; F_{cr} \tag{14}$$

where F_{cr} is the elastic critical stress, and S is the section modulus, approximated as $0.34Ad_w$. Equating Eqs.(13) and (14), the unbraced length L_c, is obtained as $138888/(d_w/A_f)F_{cr}$

A theoretical approximation for the maximum unbraced length L_c, appropriate for moment gradients is given by:

$$\frac{L_c}{r_y} = \frac{990}{\sqrt{F_y}}\left(0.93 + 0.62 \; M/M_p\right) \tag{15}$$

where r_y is the weaker axis radius of gyration and M is the moment at the adjacent brace point. This expression can be used in derivation of the second limit of Eq.(12)[9]. For $M/M_p = 0.11$ and $r_y = 0.2b_y$, this equation gives the maximum unbraced length as $200b_f$

The IS-ASD approach has no such limitation of maximum lateral ubraced length. Instead, the code lowers the allowable stress in bending compression from 160 MPa to as low as 22 MPa, depending on the ratio of overall depth of the beam to the thickness of flange, and the ratio of unbraced length of compression flange to weaker axis radius of gyration.

AISC-PD and IS-PD provisions: The limiting unbraced length L_c as per AISC-PD specifications is given by:

$$L_c \le \begin{cases} \left(\frac{9550}{\sqrt{F_y}} + 25\right) r_y & \text{for } 1.0 > \frac{M}{M_p} > -0.5 \quad (16a) \\ \left(\frac{9550}{\sqrt{F_y}}\right) r_y & \text{for } -0.5 > \frac{M}{M_p} > -1.0 \quad (16b) \end{cases}$$

Here, it is expected that the section will develop its full plastic capacity, and that the post-yield rotations can be generated, even though they are not quantified. The IS-PD approach also gives a similar expression for L_c.

$$L_c \le \begin{cases} \dfrac{960\gamma r_y}{F_y} & \text{if applied moment} > 0.85\ M_p \text{ over a length} < 40\gamma r_y \\ \dfrac{640\gamma r_y}{F_y} & \text{if applied moment} > 0.85\ M_p \text{ over a length} \ge 40\gamma r_y \end{cases} \quad (17)$$

where $\gamma = 1.5/\sqrt{1 + \theta/8}$ in which θ is the ratio of rotation at hinge point to the relatve elastic rotation of the far end of the beam segment. However, this clause requires results from detailed analysis.

AISC-LRFD Provisions: The AISC-LRFD specifications give the limiting value of unbraced length L_c, for pure elastic behaviour as:

$$L_c = \frac{r_y X_l}{F_y - F_r} \sqrt{1 + \sqrt{1 + X_2 (F_y - F_r)^2}} \quad (18)$$

where $X_l = \dfrac{\pi}{S}\sqrt{\dfrac{EGJA}{2}}$, $X_2 = \dfrac{4C_2}{I_y}\left(\dfrac{S}{GJ}\right)^2$

The maximum length of unbraced flange L_c to form a plastic hinge in the member is given by AISC-LRFD specification, as:

$$L_c = 790 r_y/\sqrt{F_y} \quad (19)$$

However, providing this maximum unbraced length does not assure that the plastic hinge can develop any further post-yield plastic rotation[9].

The AISC-LRFD specifications also provide the following expression for maximum unbraced length, which accounts for the effect of moment gradient:

$$L_c = \frac{25000 + 15278(M/M_p)}{F_y} r_y \quad (20)$$

where M is the moment at the adjacent brace point. For $M/M_p = -1.0$ and $M/M_p = -0.5$. Eq.(20) gives the unbraced length as $38r_y$ and $69r_y$, respectively. These values compare well with the values of the AISC-PD specifications.

However, under strong earthquake ground shaking, when the post-yield strain could be as high as 10 times the yield strain (even higher), the AISC-LRFD specifications require that the maximum unbraced length, L_c shall not exceed.

$$L_c = 385\ r_y/\sqrt{F_y} \quad (21)$$

This value comes to about $24r_y$. Clearly, a much more stringent control is required to cover the case of seismic condition.

SUMMARY AND CONCLUSIONS

With the development of LRFD approach for design of steel structures, the earlier design methods like ASD and PD, were also modified in the codes of USA, to account for stability considerations and made consistent with the LRFD approach. The earthquake resistant design of steel structures, is strongly influenced by the stability limit state; however, the IS-ASD specifications do not recognise this. The IS-PD spcifications on the other hand, are better for earthquake resistant design of steel structures, as these include some stability requirements. Some of the provisions in IS-PD are not adequate to sustain high strain levels generated during strong ground shaking. Also, some of these are difficult to implement, as they require data from detailed plastic analysis of the structure. Further, the following aspects are not addressed by the current Indian codes: the strength hierarchy of beam, column and joint panel zone: design and detailing of joint panel zone; and the effect of residual stresses resulting in an early yielding of extreme fibres of a beam-column section.

The salient features of the American and Indian codes pertaining to seismic design of steel structures are:

1. The American codes use the basic structural configurations with additional provisions to improve their ductility and make them earthquake resistant. In India, the ASD approach does not differentiate the design for gravity forces from that for seismic forces.

2. A few of the stability provisions existing in the American codes, are available in Indian code only as part of the plastic design approach. It is desirable that IS-ASD approach should incorporate them from IS-PD approach along with the other stability provisions given in the American codes.

3. The concept of compact and non-compact sections should be introduced in the Indian design practice. This will serve as a preliminary platform for understanding the necessity of the stability criterion.

4. In earthquake resistant construction, the column and beam sections should be of compact type, to be able to develop their full strength, and further they should be able to sustain high post-yield strain. The seismic requirement of unsupported width-to-thickness ratio of beam and column flanges is 8.5. Almost all Indian standard hot-rolled sections satisfy this criterion, (Table 3). However, attention should be paid while designing non-standard sections, e.g., plate girders, with wide flanged sections or welded plate sections for seismic conditions. The IS-ASD approach allows a maximum unsupported width-to-thickness ratio of beam and column flanges as 16 for steel grade Fe250, which may not be adequate for structures in high seismic zones.

5. American design practice has clear guidelines regarding the maximum unbraced length of compresion flange of beams, corresponding to elastic, inelastic and

plastic strain levels to avoid lateral-torsional buckling. The IS-ASD approach instead lowers the allowable stress in bending compression to compensate for the large unbraced length. On the other hand, IS-PD approach has some guidelines to estimate the maximum unbraced length, but these are too complicated for practical application. Under strong seismic shaking, the American design practice proposes a maximum unbraced length of 24 times of the weaker axis radius of gyration. Which is also difficult to implement from cosntruction point of view.

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