Limitations of available Indian Hot-Rolled I-Sections for use in Seismic Steel MRFs

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Abstract

Steel hot rolled I-sections have been in use in construction since long in India. With advancement of technology to build moment resisting frames (MRFs) to resist seismic actions, a review of the existing available sections is required to assess their applicability. This paper reiterates the important aspects of the seismic design philosophy and investigates the available sections in light of the same. The sectional properties (strength and stability) are studied in light of the different code requirements for desired performance under strong seismic conditions. Indian hot-rolled I-sections (tapered and parallel flanges) are found inadequate for use in tall structures in high seismic regions.

1. Introduction

Satisfactory performance of steel structures in high seismic regions depends on numerous factors. Three significant factors in design are stability, strength and ductility of individual members. Apart from these, connections play an important role in the overall performance of the structure; inadequate connections can result in failure of the structure even if the structural members are adequately designed. A proper design considering these, together with a satisfactory collapse mechanism under strong seismic shaking results in good overall performance of the structure. In this paper, the international state-of-the-art seismic design provisions for steel sections are reviewed. The limited range of hot-rolled steel I-sections available in India for steel construction are evaluated to identify the suitability of their use in high seismic environment.

2. Strength Criteria and Capacity Design Philosophy

In the past few decades, the evolution of the *Capacity Design* concept is one of the most important developments in the field of earthquake-resistant design of structures [*e.g.,* Paulay and Priestly, 1992]. Through this concept, structures can be designed to behave in a pre-determined manner during strong earthquake shaking. This includes, most importantly, preventing brittle types of failure and forcing ductile action in the structural components. Also, while ensuring that a pre-determined desired mechanism occurs (for instance, beam sway mechanism is preferred over storey mechanism in

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multistorey building MRFs), the most common design practice evolved, namely the *strong-column weak-beam* approach of proportioning frame members is used. Further, following the large number of connection failures in steel MRFs during the 1994 Northridge earthquake (USA) and 1995 Kobe earthquake (Japan), the seismic design of beam-to-column connections now requires that these connections be designed as per the capacity design concept. In summary, the capacity design concept enlists a strength hierarchy of the components of a building: (a) the beam-to-column connections joint are to be stronger than the beam, (b) the columns are to be stronger than the beams, and (c) the column base connections are to be stronger than the column [Penelis and Kappos, 1997].

In the above consideration of the earthquake-resistant design philosophy, estimation of the maximum strength that is achievable in a member (beam/column) under strong earthquake shaking is important. This strength, called the *overstrength* capacity, is more than the nominal strength of the members obtained using the code-specified design procedures. Overstrength occurs due to redundancy in the structural system, the partial safety factors for materials, and differences in the actual and idealized stress-strain curves of materials. Two factors related to the last aspect causing material overstrength are discussed in the following.

2.1 Yield Strength of Material

The existing code procedures for the design of steel members are based on the minimum specified characteristic yield strength f_y. However, coupon tests have shown that the actual yield strength of material are often higher than the minimum specified yield strength [Engelhardt and Sabol, 1998; Malley and Frank, 2000]. This causes an increase in actual member strength over that estimated using code-prescribed procedures. AISC [AISC, 2002] indicates that the ratio of the expected yield strength to the minimum specified characteristic yield strength, herein named R_y, varies between 1.1 to 1.3 depending on the grade of steel. In India, such data for the available Indian sections are not readily available in public literature, and also, the current code provisions do not account for this. Such statistical data from the Indian hot-rolled sections obtained through coupon test need to be incorporated in seismic design procedures.

2.2 Strain Hardening of Steel

The Indian steel code [IS:800, 1984] assumes an idealized elastic perfectly-plastic constitutive law for structural steel with characteristic yield strength as f_y. In reality, structural steel has a distinct constitutive relation (Figure 1) with an initial elastic zone (OA), a yield plateau (AB), a strain-hardening zone (BC), and a strain-softening zone (CD) before it fractures. The member nominal flexural strength, *i.e.*, plastic moment capacity M_p for bending about the major axis, is computed based on the idealized rectangular stress block with a maximum stress of f_y. Such a stress block is not practically achievable, because to develop a stress of f_y at the fibers at and near the neutral axis, the strains required at the extreme fibers of the section are infinitely large. Secondly, the rectangular stress block can never be achieved without strain-hardening of the extreme fibers of the beam section. Thus, the representation of M_p using rectangular stress blocks deviates from the actual behavior.

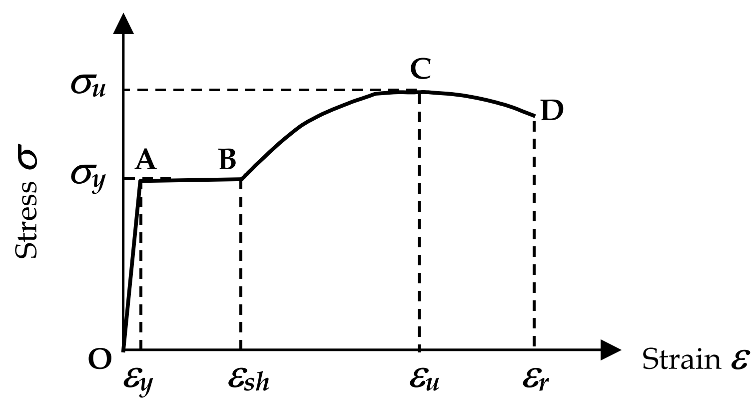


Figure 1: *Typical schematic of constitutive curve of structural steel:* Four distinct zones are evident - a linear elastic zone OA, a yield plateau AB, a strain-hardening zone BC and a strain-softening zone CD.

The beam bending moment equal to the plastic moment value M_p can be realized in a section only when a part of it undergoes strain-hardening while some of it still remains elastic (Figure 2). Thus, the beam design based on M_p indirectly accounts for only a marginal amount of strain-hardening. However, although the maximum capacity of the beam corresponding to the ultimate stress f_u in the extreme fiber may never be achieved (as the associated curvature ductility demands of around *100* and deformations required to accommodate such large curvatures are impractical); recent experimental studies [Englehardt and Sabol, 1998] show that beam capacities larger than M_p are definitely achievable with inelastic deformations corresponding to the drift demands expected by some code guidelines [UBC, 1997; FEMA, 1995]. Thus, it is the strain-hardening of steel that causes an increase in the member capacity under strong seismic shaking over the code-prescribed nominal capacity M_p.

Curvature ductility μ imposed at a section can be estimated from the amount of plastic rotation θ_p required to be developed at the end of the member, using [*e.g.*, Arlekar and Murty, 2000b]

$$\mu = \frac{2EI\theta_p}{M_p d}, \tag{1}$$

where d and EI are the depth and flexural rigidity of the member. The AISC code recommended plastic rotation demand θ_p varies between 0.01 and 0.04 radians. Since Indian steel code specifies no such demand, experiments need to be conducted on MRFs made using Indian Standards sections to determine desirable plastic rotations and develop associated design guidelines.

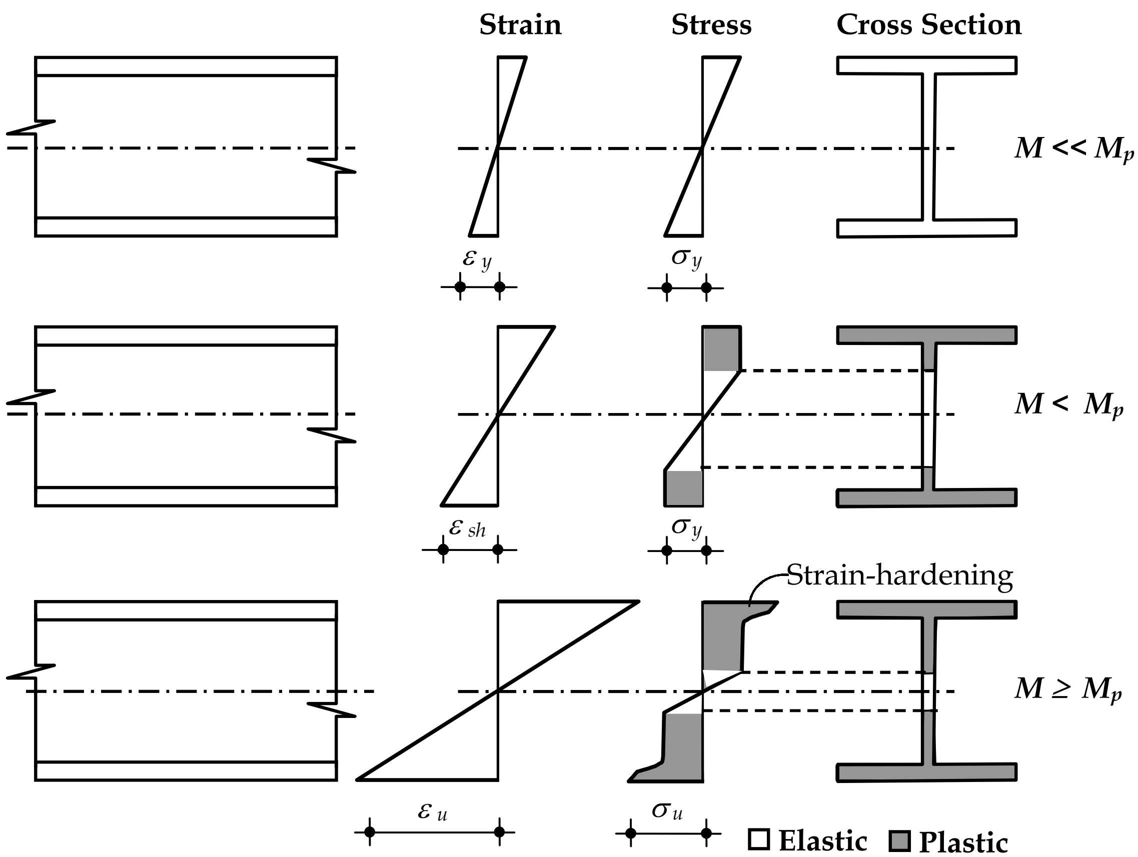


Figure 2: *Member Plastification:* Various stages of member plastification under pure flexure (adapted from Bresler and Lin, 1960). The fibers at neutral axis do not yield, but the fibers away from the neutral axis strain-harden, thus the cross-section develops a moment of M_p and more.

Sixty one hot-rolled Indian standard I-sections are considered in this work to study the effect of strain-hardening on section capacity (Table 1). Figure 3 shows the variation of normalized moment M/M_p developed as a function of curvature ductility $\mu = (\varphi/\varphi_y)$ imposed at the cross-section for the hot-rolled Indian I-sections. These curves are generated for f_y = 250*MPa* and $f_u/f_y = 1.5$ using a fiber model described in another paper [Goswami *et al,* 2003]. The shape of these curves imitates the stress-strain curve of steel as shown in Figure 1. The (M/M_p) *versus* μ curves of the sixty one sections are so close to each other (Figure 3) that they can be idealized by a single curve having elastic, perfectly-plastic and smooth strain-hardened regions given by the following:

$$\frac{M}{M_p} = R_s = \begin{cases} \mu & \text{for } 0 \le \mu \le \mu_y \\ 1 & \text{for } \mu_y < \mu \le \mu_{sh} \\ 0.81 + 2\left(\frac{\mu}{100}\right) - 2\left(\frac{\mu}{100}\right)^2 + \left(\frac{\mu}{100}\right)^3 - 0.3\left(\frac{\mu}{100}\right)^4 & \text{for } \mu_{sh} < \mu \le \mu_u \end{cases} , \qquad (2)$$

where μ_y is the curvature ductility at idealized yield, μ_{sh} is the curvature ductility at the beginning of strain-hardening on the idealized curve, and μ_u is the ultimate curvature ductility. From the data of the 61 sections considered, the values of μ_y, μ_{sh} and μ_u are obtained as 1.0, 11.4 and 150 respectively. Using Eq.(1), the curvature ductility μ of the sections considered ranges from 7.0 to 29.0 for θ_p varying between 0.01 and 0.04 radians (as noted in AISC code). Using this and Eq.(2), the value of R_s, hereinafter called the *strain-hardening factor*, is estimated to be in the range 1.0 to 1.24.

3. Section Geometry

An important feature of the generally available Indian hot-rolled I-sections is their tapered flanges. Due to the tapering, bolt-shank bends on tightening, thereby increasing the chances of its failure. Also, because of the tapered and thin tip of the flange, only small size welds are possible between the cover plate and the flange tip (Figure 4). Moreover, proper welding between surfaces at such obtuse angle is difficult, and again increases chance of brittle failure of the weld. Another concern is the small flange width of the sections; the largest flange in all sections is only 250*mm*. Apart from offering low strength and stiffness, the small flange width allows the use of only one bolt on either side of the web and therefore requires unduly large connection length.

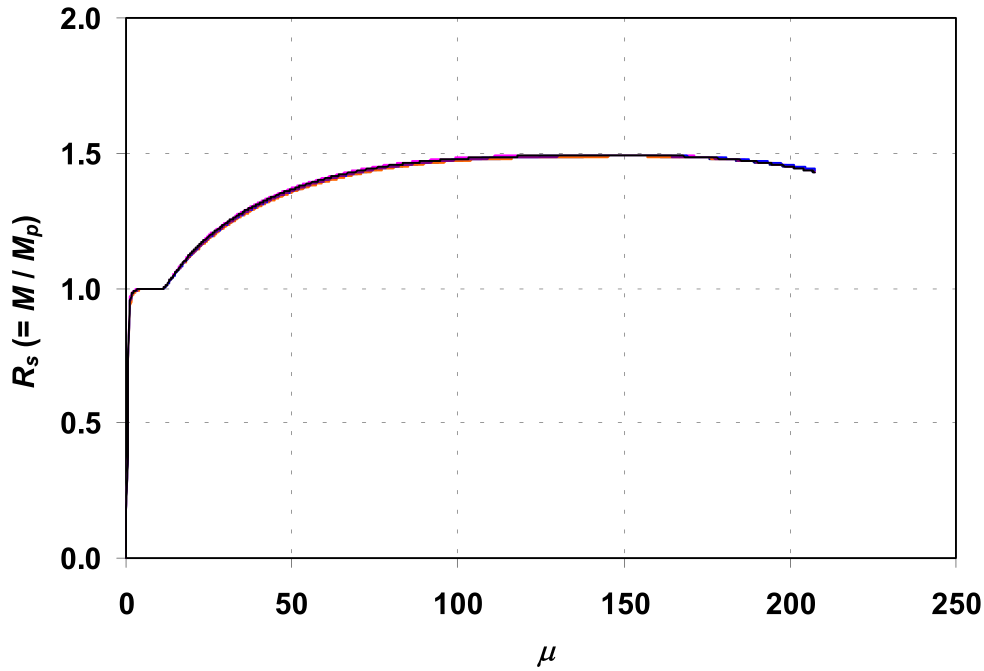


Figure 3: Beam moment developed for $R_y = 1.5$ at different levels of curvature ductility imposed on the Indian I-sections considered in this study.

Poor and unreliable welding in welded connection scheme and large connection length in bolted connection scheme puts the cover-plated connections of Indian hot-rolled sections with taped flanges in jeopardy. Thus, in summary, the tapered flanges of the Indian hot-rolled I-sections pose many difficulties. For this reason, countries with advanced provisions in seismic design of steel structures, like the USA, only use hot-rolled sections with uniform thickness flanges.

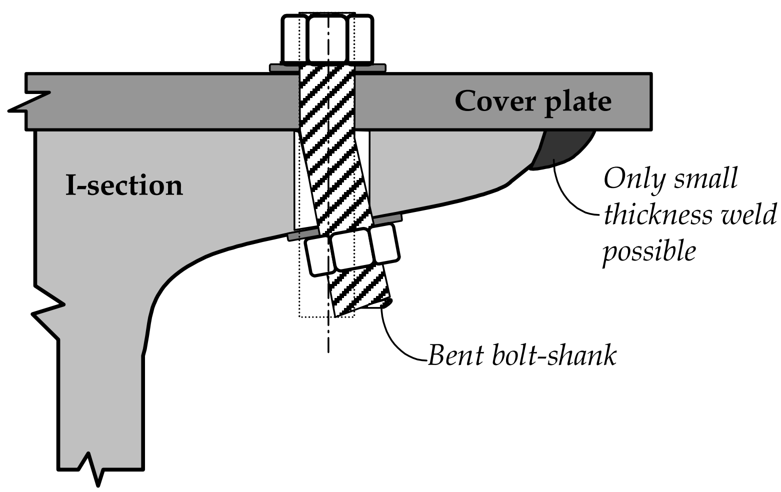


Figure 4: *Effects of tapered flange*: (i) Bolted connection: Bolt shank gets bent on tightening from the original straight alignment and (ii) Welded connection: Only obtuse angled small thickness weld possible at the tapered tip.

Considering the difficulties associated with construction and behaviour of the tapered flange I-sections, hot rolled steel sections with parallel flanges with square toes and curves at the root of the flange and web are now gradually being produced in India. Recently, the Bureau of Indian Standards has taken initiative to revise IS 12778 [IS 12778, 2003], which includes section dimensions of such parallel flange sections.

4. Stability Criteria

Local buckling of flanges and web of a member can adversely affect its maximum strength. On the basis of maximum inelastic deformation and ultimate strength achieved, sections are grouped under three heads namely, *compact*, *semi-compact* and *slender*. The deformation and strength capacity of sections, and of members as a result, is usually limited by effect of instability. In steel I-sections subjected to flexure, the different forms of instability are: (a) flange local buckling (FLB), (b) web local buckling (WLB), (c) lateral torsional buckling (LTB), and (d) overall column buckling [Bruneau *et al*, 1998]. The design codes uses slenderness or b/t ratios to identify stability limits of flange and web plates. From AISC codes [AISC 1989, AISC 1994, AISC 1997], these limits can be taken as: (a) λ_{pd} - slenderness limit for compact elements with a minimum guaranteed ultimate strength M_p and plastic rotation ductility, (b) λ_p - slenderness limit for compact elements with only minimum guaranteed strength M_p, and (c) λ_r - slenderness limit for non-compact elements with only minimum guaranteed strength M_y (Figure 5). Structural members with flanges and web elements classified as slender $(\lambda > \lambda_r)$ buckle locally even before reaching their yield moment capacity M_y, while structural members with non-compact elements $(\lambda_p < \lambda < \lambda_r)$ are able to reach the yield moment only. Structural members with compact elements $(\lambda_{pd} < \lambda < \lambda_p)$ are able to develop the member plastic capacity M_p with limited ductility while members with elements with b/t limits less that λ_{pd} develop full member plastic capacity M_p and sufficient plastic rotation.

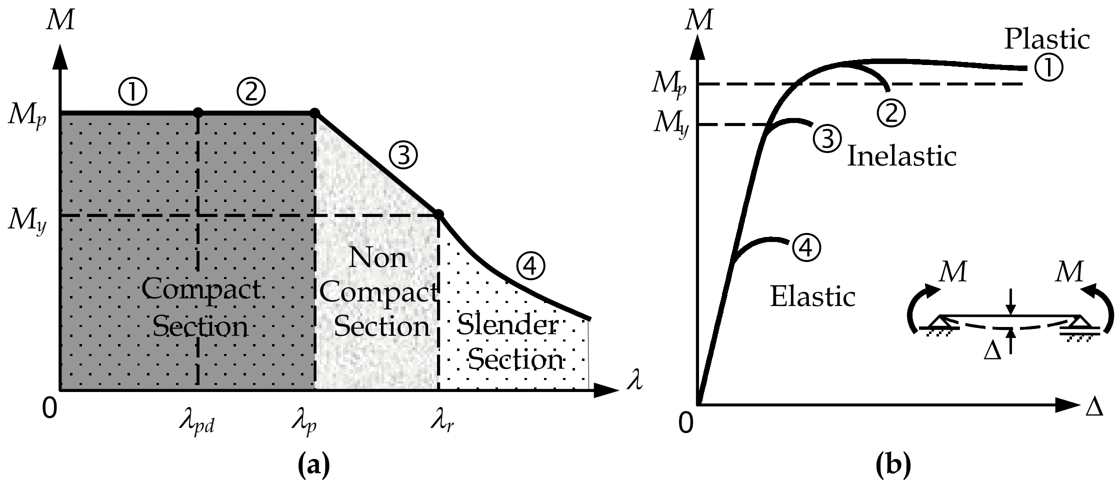


Figure 5: *Effect of slenderness on developable member capacity*: (a) Strength-slenderness ratio relationship; (b) Moment-deflection behavior of I-sections, for different levels of slenderness. Inelastic buckling commences much before yield moment M_y is reached because of residual stresses.

The Indian Standard Handbook [SP:6(1), 1964] classifies Indian hot-rolled I-sections into four categories namely, *light* (ISLB), *medium* (ISMB), *wide flange* (ISWB) and *heavy* (ISHB). These have the unique feature that the flanges are tapered with rounded corners at the ends. The $\left(b_f / t_f\right)$ and $\left(d_w / t_w\right)$ ratios of these different sections are shown in Figures 6 and 7, respectively. In these figures, the limits of b_f / t_f and d_w / t_w ratios for beam and column flanges and webs as prescribed in Allowable Stress Design Method and Plastic Design Method in Indian Standard [IS 800, 1984], Load and Resistance Factor Design Method in AISC [AISC, 1999] and Seismic Provisions for Structural Steel Buildings in AISC [AISC, 1999] are also shown for comparison.

The following discussion uses $f_y = 250MPa$. The IS-ASD limits the maximum unsupported flange width-to-thickness ratio to $256 \big/ \sqrt{f_y}$, *i.e.,* to 16.2. Similarly, the prescribed maximum web depth-to-thickness ratio is 85. On the other hand, the IS-PD prescribes a flange width-to-thickness ratio as $136 \big/ \sqrt{f_y}$, *i.e.*, as 8.6, and maximum web depth-to-thickness ratio as $688 \big/ \sqrt{f_y}$, *i.e.*, as 43.5 for P / P_y exceeding 0.27. For P / P_y less than 0.27, the maximum web depth-to-thickness ratio is given by

$$\frac{1120}{\sqrt{f_y}}\left(1 - 1.43\frac{P}{P_y}\right) \qquad \text{for } \frac{P}{P_y} \leq 0.27 \tag{3}$$

giving a value of $1120/\sqrt{f_y}$, *i.e.*, 70.8 for no axial stress. Here, P and P_y are the design and yield load of the compression member.

The AISC-LRFD provisions recommend a maximum flange width-to-thickness ratio of $170/\sqrt{f_y}$, *i.e.*, 10.7 and $355/\sqrt{f_y}$, *i.e.*, 22.4 for compact and non-compact sections respectively. Similarly, for compact sections, the maximum web depth-to-thickness ratio is recommended as

$$\frac{500}{\sqrt{f_y}}\left(2.33 - \frac{P_u}{\phi P_y}\right) \geq \frac{666}{\sqrt{f_y}} \qquad \text{for } \frac{P_u}{\phi P_y} > 0.125 \text{, and} \tag{4}$$

$$\frac{1680}{\sqrt{f_y}}\left(1 - \frac{2.75 P_u}{\phi P_y}\right) \qquad \text{for } \frac{P_u}{\phi P_y} \leq 0.125; \tag{5}$$

giving a range of $666/\sqrt{f_y}$, *i.e.*, 42.1 for $P_u/\phi P_y = 1$ and increasing to $1680/\sqrt{f_y}$, *i.e.*, 106.2 for $P_u = 0$. Here, P_u is the factored axial load on the compression member and ϕ is the strength reduction factor. For non-compact sections, the maximum web depth-to-thickness ratio limit is set as

$$\frac{2250}{\sqrt{f_y}}\left(1 - \frac{0.74 P_u}{\phi P_y}\right), \tag{6}$$

giving a range of $663/\sqrt{f_y}$, *i.e.*, 41.9 for $P_u/\phi P_y = 1$ to $2250/\sqrt{f_y}$, *i.e.*, 142.3 for $P_u = 0$.

The AISC-SPSSB specifications recommend, for seismically compact sections, a maximum flange width-to-thickness ratio of $134/\sqrt{f_y}$, *i.e.*, 8.5 for beams and $170/\sqrt{f_y}$, *i.e.*, 10.7 for columns. The maximum web depth-to-thickness ratio is given as

$$\frac{500}{\sqrt{f_y}}\left(2.33 - \frac{P_u}{\phi P_y}\right) \geq \frac{666}{\sqrt{f_y}} \qquad \text{for } \frac{P_u}{\phi P_y} > 0.125 \text{, and} \tag{7}$$

$$\frac{1405}{\sqrt{f_y}}\left(1 - \frac{1.54 P_u}{\phi P_y}\right) \qquad \text{for } \frac{P_u}{\phi P_y} \leq 0.125; \tag{8}$$

giving a range of $666/\sqrt{f_y}$, *i.e.*, 42.1 for $P_u/\phi P_y = 1$ and increasing to $1405/\sqrt{f_y}$, *i.e.*, 88.8 for $P_u = 0$. A detailed discussion on these different provisions is provided elsewhere [Paul *et al.*, 2000].

The slenderness ratio, the flange width-to-thickness ratio and the web depth-to-thickness ratio of the Indian I-sections are compared against the above code-prescribed limiting values. Using the AISC-LRFD categories of *compact* and *non-compact* sections and the AISC-SPSSB category of *seismic* sections, it is seen from Figure 6 that, barring ISHB 200 to ISHB 450, and ISWB 250 and ISWB 300 which do not conform to seismic criterion with respect to flange width-to-thickness ratio if is to be used as beams, all other sections are compact. From Figure 7, based on web depth-to-thickness ratio, it is seen that, in general, all sections of depth up to 300mm conform to seismic criterion, with higher ones generally conforming to the requirements for design axial loads not exceeding about 60% of the axial capacity. For the parallel flange sections, from Figures 8 and 9, it is seen that although most of the sections are compact with respect to flange and web plate slenderness limits, still some do not conform to the criteria even that of the Indian standard.

Further, apart from the section compactness, member stability is another important aspect ensuring satisfactory performance of the final designed structure. In absence of lateral support against bending about their weaker axis, almost all of these sections fail to comply with required member slenderness for columns under strong seismic action because of their small radius of gyration; additional flange plates are required if these sections are to be used in MRFs intended to resist seismic actions [Paul *et al.*, 2000a].

In addition, local buckling can occur in Indian hot-rolled I-sections at low post-yield strains due to presence of *residual stresses*. Material non-linearity was shown to begin at about 70 to 43 percent of the plastic moment capacity for residual stresses of 70*MPa* and 140*MPa* respectively. Consequently, flexural plastic capacity is reached at extreme fibre strain of about 2.4 to 2.8 times the yield strain. This high strain can cause local buckling [Paul *et al.*, 1999]. All these aspects raise the concern on the stability of structures built using the available Indian hot-rolled I-sections with tapered flanges for resisting earthquake effects.

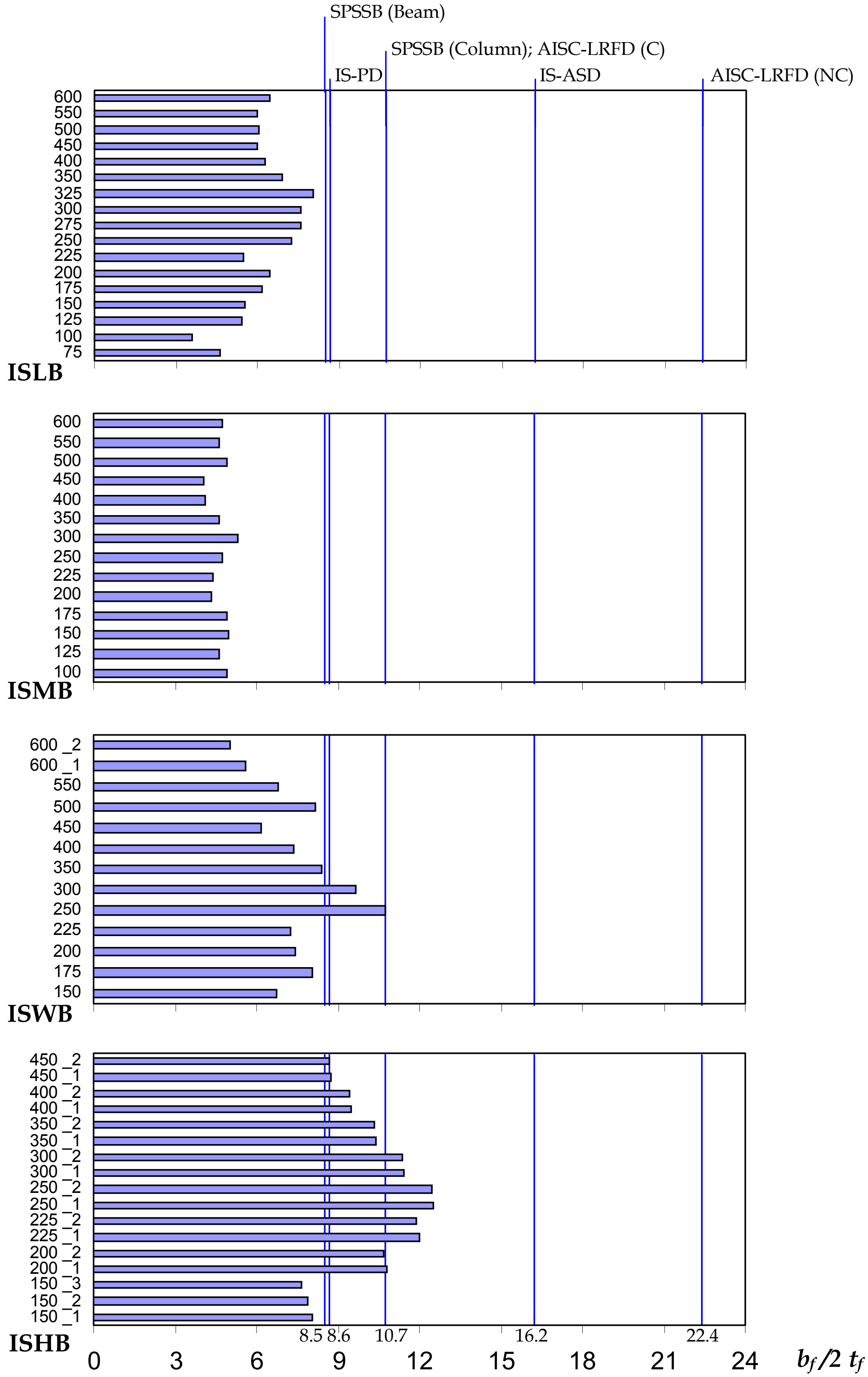


Figure 6: *Flange width-to-thickness ratio of Indian hot-rolled tapered flange I-sections:* All ISLB and ISMB sections comply with requirements for seismic condition.

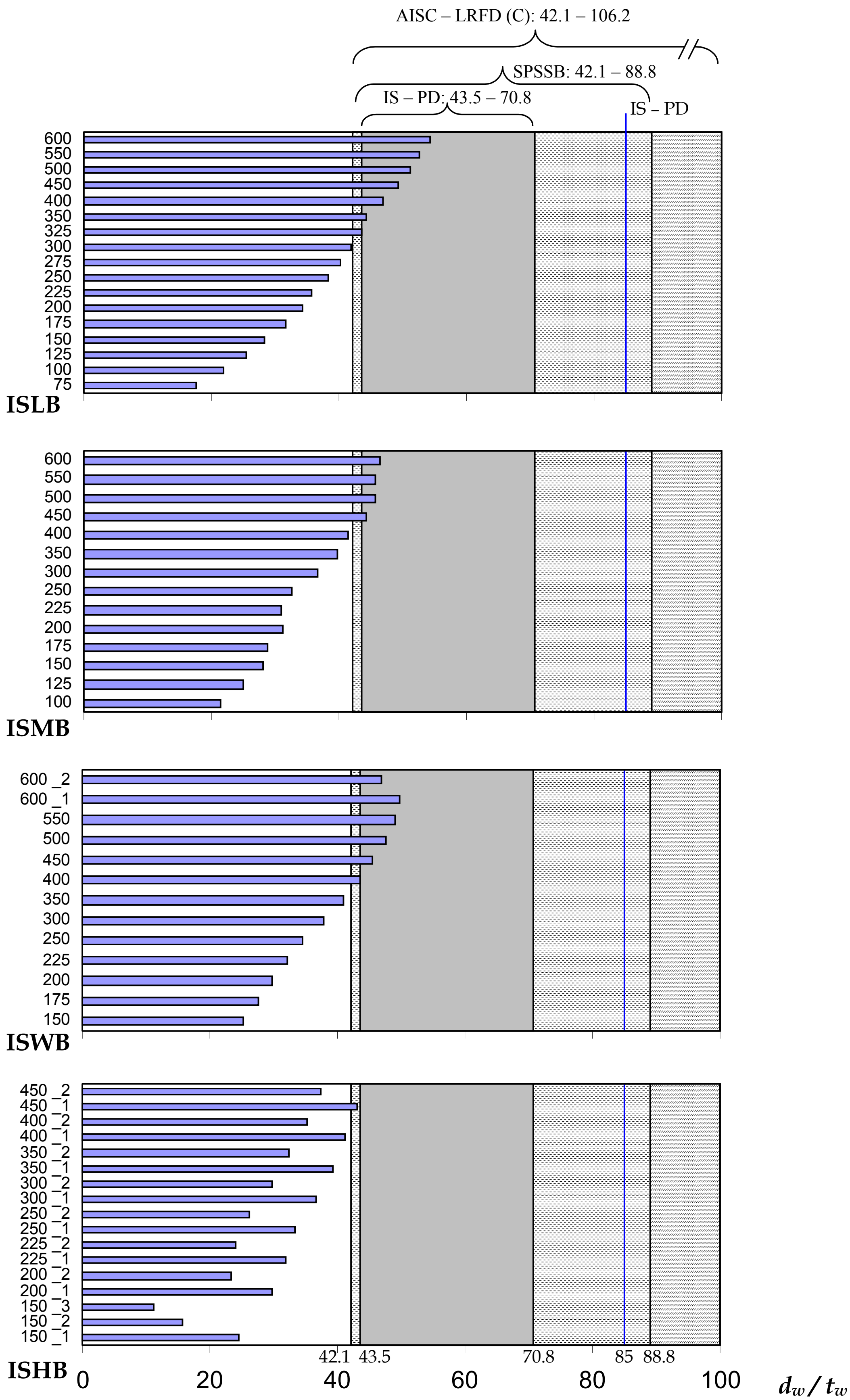


Figure 7: *Web depth-to-thickness ratio of Indian hot-rolled tapered flange I-sections:* All ISLB and ISMB sections comply with requirements for seismic condition.

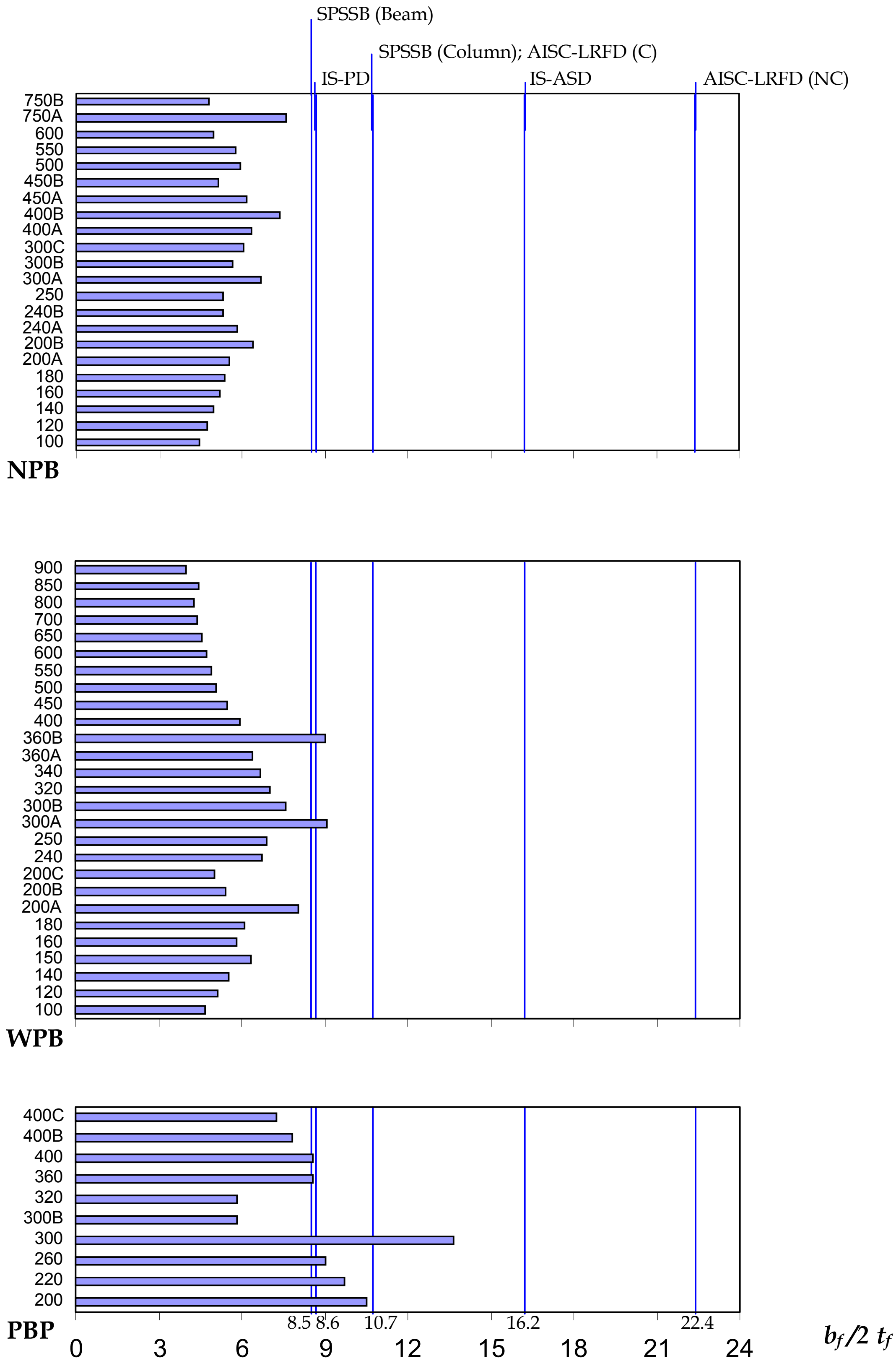


Figure 8: *Flange width-to-thickness ratio of Indian hot-rolled parallel flange sections:* All NPB sections comply with requirements for seismic condition.

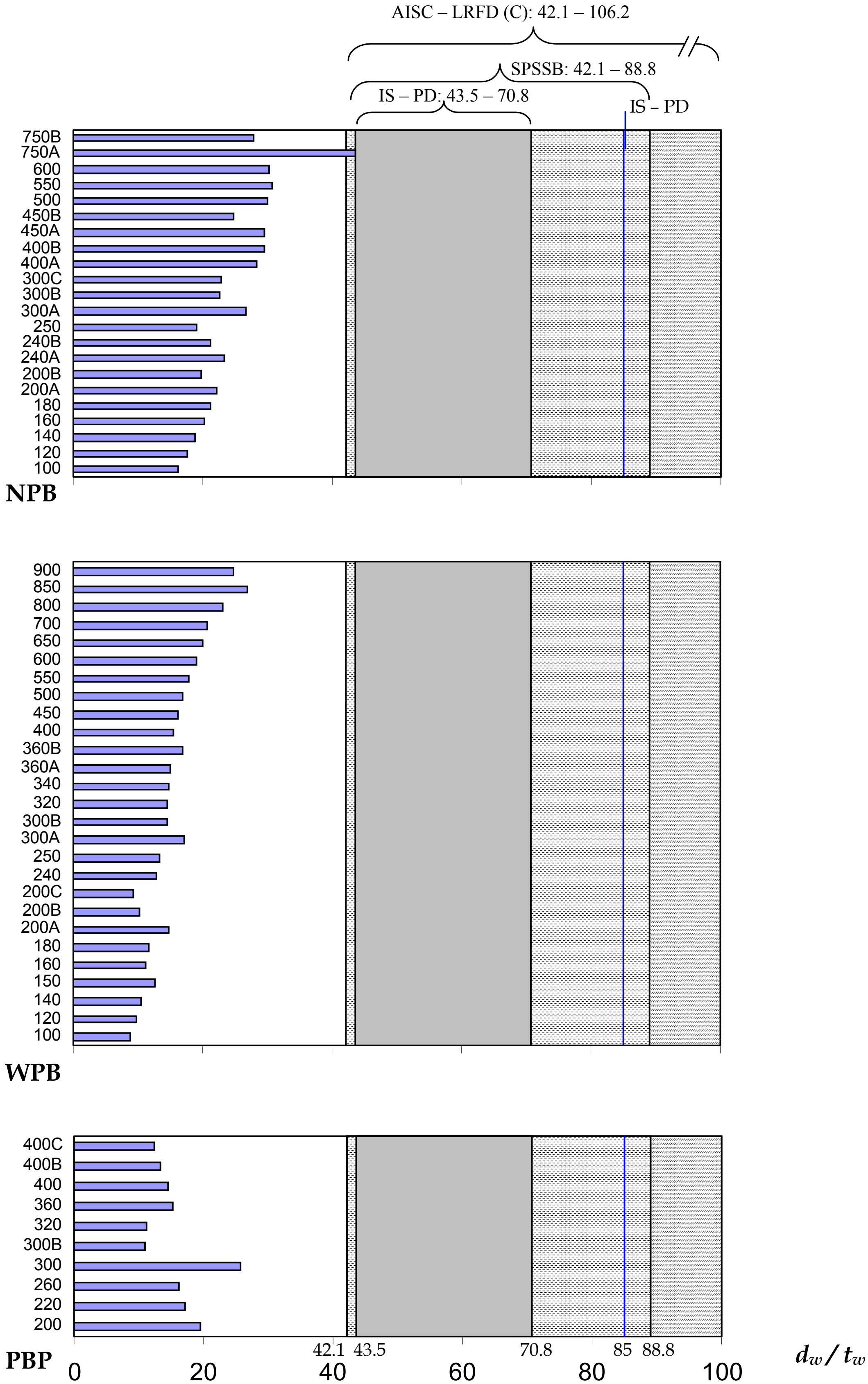
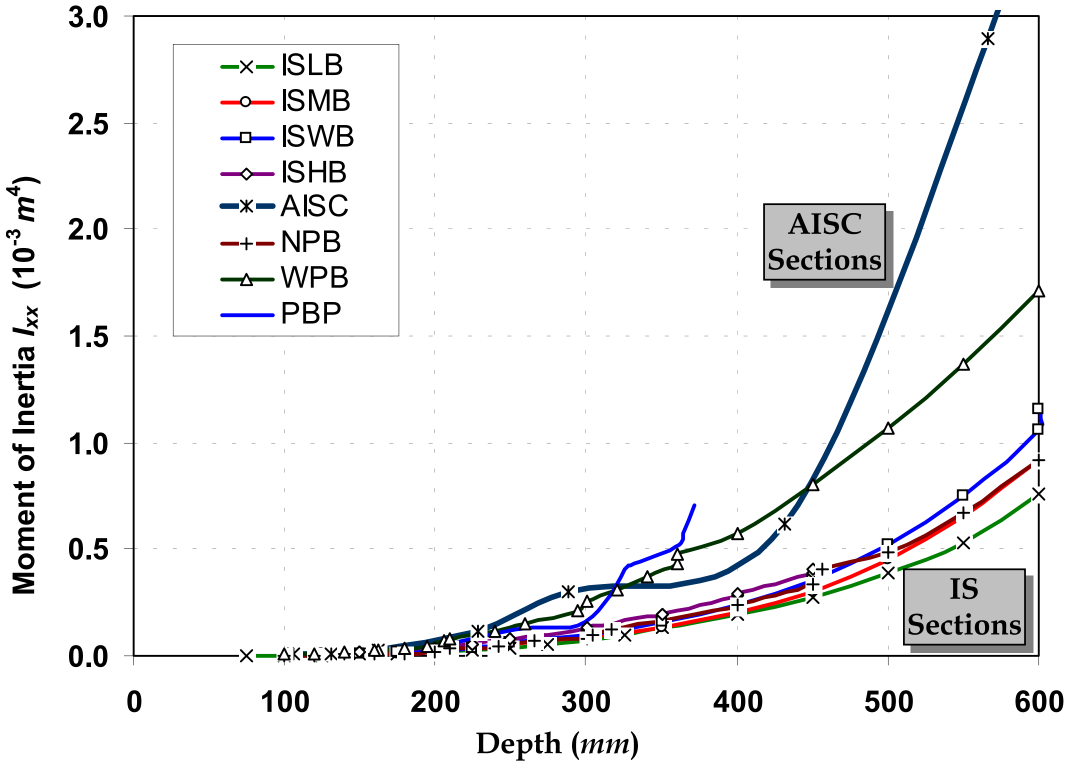


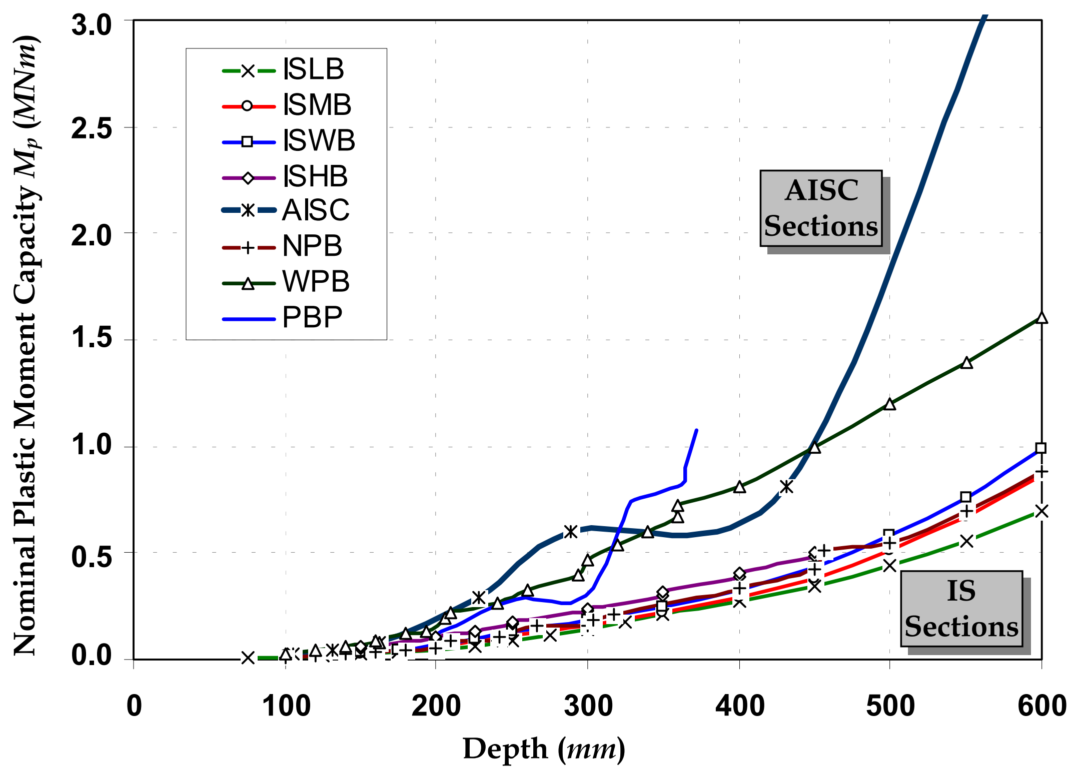
Figure 9: *Web depth-to-thickness ratio of Indian hot-rolled parallel flange sections:* All WPB and PBP sections comply with requirements for seismic condition.

5. Stiffness and Strength

The following is a comparison of the stiffness and strength of some representative IS sections (tapered and parallel flanges) with representative AISC sections commonly used in earthquake-resistant construction in the USA. Tables 1 and 2 list the properties of the Indian I-sections, while Table 3 lists the properties of the representative AISC sections used in this study. The maximum depth of Indian I-sections with tapered flange is 600mm (for sections ISLB 600, ISMB 600, ISWB 600). The section properties given in SP 6(1) [SP6(1), 1964] suggest that the highest moment of inertia (I_{xx}) is that of ISWB 600 followed by ISMB 600. Also, the nominal plastic moment capacity (M_p) is largest for these two sections. For IS sections with parallel flanges, the maximum depth is 900mm for WPB 900x300x291.45. Consequently, it also has the largest moment of inertia and plastic moment capacity between the available hot-rolled Indian sections with parallel flanges. However, the moment of inertia and the stiffness of AISC sections are still about 2 to 3 times higher than those of the Indian sections of same depth (Figure 8a). The difference is even higher in case of nominal plastic moment capacities; the AISC sections have 2 to 4 times larger M_p than those of ISMB sections of same depth while the NPB and the PBP sections compete to some extent (Figure 8b). Moreover, the depths of available Indian sections are still small for use in tall earthquake-resistant structures (Figure 9). Also, the flange widths of the Indian sections are small; WPB 900x300x291.45 has a flange width of only 300mm. In other words, the strength and stiffness of Indian sections are too low to be satisfactorily used in earthquake-resistant design of tall structures; only low-rise constructions may be possible.



(a)



(b)

Figure 8: *Comparison of section properties of representative AISC and IS hot-rolled I-sections:* (a) Difference of moment of inertia of sections; (b) Difference of nominal plastic moment capacity of sections. Indian sections are much smaller than the AISC sections.

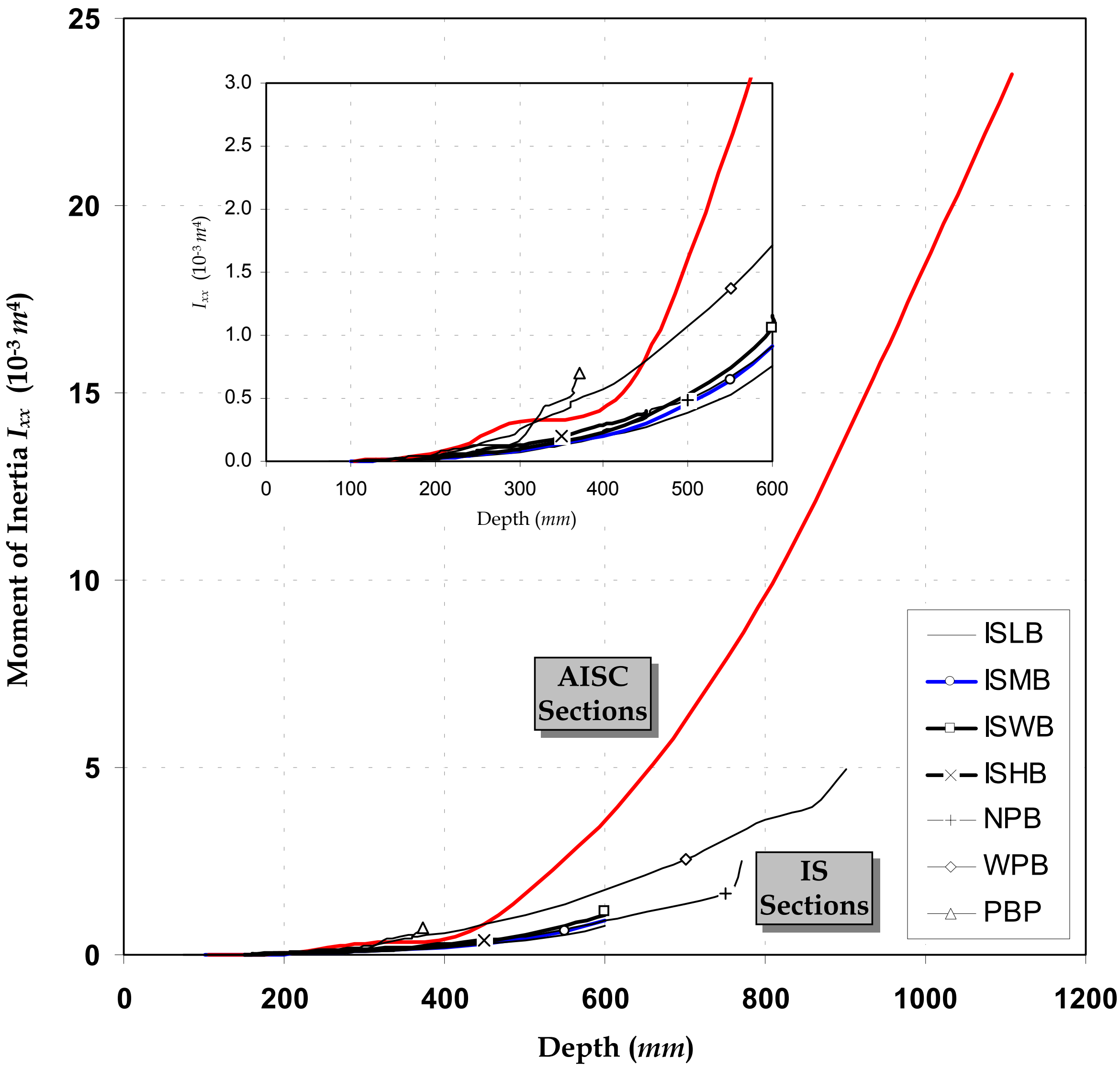


Figure 9: *Comparison of section properties of representative AISC and IS hot-rolled I-sections with tapered and parallel flanges:* Difference of moment of inertia of sections. Maximum depth of Indian section is 900mm while that of ASTM sections is around 1100mm. However, Indian sections are smaller and have much smaller moment capacity than the AISC sections.

Table 1: Moment of inertia and nominal plastic moment capacity of Indian I-sections.

Section	Depth d (*mm*)	I_{xx} (10^{-6} m^4)	M_p (*kNm*)	Section	Depth d (*mm*)	I_{xx} (10^{-6} m^4)	M_p (*kNm*)
ISLB 75	75	0.73	5.39	ISMB 100	100	2.58	14.49
ISLB 100	100	1.68	9.36				
ISLB 125	125	4.07	18.01	ISMB 125	125	4.49	20.29
ISLB 150	150	6.88	25.38	ISMB 150	150	7.26	27.33
ISLB 175	175	10.96	34.86	ISMB 175	175	12.72	41.16
ISLB 200	200	16.97	47.14	ISMB 200	200	22.35	62.60
ISLB 225	225	25.02	63.50	ISMB 225	225	34.42	86.01
ISLB 250	250	37.18	84.92	ISMB 250	250	51.32	114.85
ISLB 275	275	53.75	111.27				
ISLB 300	300	73.33	138.85	ISMB 300	300	86.04	161.59
ISLB 325	325	98.75	172.83				
ISLB 350	350	131.58	212.18	ISMB 350	350	136.30	219.57
ISLB 400	400	193.06	273.45	ISMB 400	400	204.58	290.53
ISLB 450	450	375.36	348.14	ISMB 450	450	303.91	382.98
ISLB 500	500	385.79	440.32	ISMB 500	500	452.18	512.80
ISLB 550	550	531.62	553.69	ISMB 550	550	648.94	670.49
ISLB 600	600	758.68	696.58	ISMB 600	600	918.13	867.80
ISWB 150	150	8.39	31.52	ISHB 150_1	150	14.56	53.38
				ISHB 150_2	150	15.40	56.82
				ISHB 150_3	150	16.36	60.53
ISWB 175	175	15.09	48.36				
ISWB 200	200	26.25	73.02	ISHB 200_1	200	36.08	98.88
				ISHB 200_2	200	37.22	102.38
ISWB 225	225	39.21	96.87	ISHB 225_1	225	52.80	128.50
				ISHB 225_2	225	54.79	134.09
ISWB 250	250	59.43	131.77	ISHB 250_1	250	77.37	169.19
				ISHB 250_2	250	79.84	175.50
ISWB 300	300	98.22	182.27	ISHB 300_1	300	125.45	229.34
				ISHB 300_2	300	129.50	238.06
ISWB 350	350	155.22	247.94	ISHB 350_1	350	191.60	301.55
				ISHB 350_2	350	198.03	313.53
ISWB 400	400	234.27	327.97	ISHB 400_1	400	280.84	388.74
				ISHB 400_2	400	288.24	402.67
ISWB 450	450	350.58	435.81	ISHB 450_1	450	392.11	485.50
				ISHB 450_2	450	403.50	502.20
ISWB 500	500	522.91	585.09				
ISWB 550	550	749.06	760.76				
ISWB 600_1	600	1061.99	987.77				
ISWB 600_2	600	1156.27	---				

Table 2: Moment of inertia and nominal plastic moment capacity of some representative Indian I-sections with parallel flange.

Section Name (NPB)	Label	Depth (mm)	I_{xx} (10^{-6} m^4)	M_p (kNm)
100x55x8.10	(100)	100	1.71	9.85
120x60x10.37	(120)	120	3.18	15.18
140x70x12.89	(140)	140	5.41	22.09
160x80x15.77	(160)	160	8.69	30.97
180x90x18.80	(180)	180	13.17	41.61
200x100x22.36	(200A)	200	19.43	55.17
200x130x31.55	(200B)	210	31.53	84.30
240x120x30.71	(240A)	240	38.92	91.67
240x120x34.31	(240B)	242	43.69	102.58
250x150x46.48	(250)	266	73.81	156.37
300x150x42.24	(300A)	300	83.56	157.10
300x150x49.32	(300B)	304	99.94	185.97
300x165x53.46	(300C)	317	121.23	214.40
400x180x66.30	(400A)	400	231.28	326.82
400x200x67.28	(400B)	400	242.24	338.77
450x190x77.57	(450A)	450	337.43	425.48
450x190x92.36	(450B)	456	409.23	511.60
500x200x90.69	(500)	500	481.99	548.57
550x210x105.52	(550)	550	671.16	696.81
600x220x122.45	(600)	600	920.83	878.16
750x270x145.29	(750A)	750	1619.58	1252.48
750x270x202.48	(750B)	770	2495.37	1857.76

Section Name (WPB)	Label	Depth (mm)	I_{xx} (10^{-6}	M_p (kNm)
100x100x20.44	(100)	100	4.50	26.06
120x120x26.69	(120)	120	8.64	41.31
140x140x33.72	(140)	140	15.09	61.36
150x150x36.98	(150)	162	22.10	77.19
160x160x42.59	(160)	160	24.92	88.50
180x180x51.22	(180)	180	38.31	120.37
200x200x50.92	(200A)	194	45.31	130.38
200x200x74.01	(200B)	206	71.73	198.34
200x200x83.52	(200C)	209	80.58	222.20
240x240x83.20	(240)	240	112.59	263.30
250x250x97.03	(250)	260	150.30	326.40
300x300x100.84	(300A)	294	210.46	396.08
300x300x117.03	(300B)	300	251.66	467.20
320x300x126.65	(320)	320	308.24	537.34
340x300x134.15	(340)	340	366.56	602.06
360x300x141.80	(360A)	360	431.93	670.79
360x370x150.87	(360B)	360	473.02	726.15
400x300x155.26	(400)	400	576.80	807.98
450x300x171.11	(450)	450	798.88	995.64
500x300x187.33	(500)	500	1071.76	1203.70
550x300x199.44	(550)	550	1366.91	1397.72
600x300x211.92	(600)	600	1710.41	1606.36
650x300x224.78	(650)	650	2106.16	1830.05
700x300x240.51	(700)	700	2568.88	2081.87
800x300x262.33	(800)	800	3590.83	2557.30
850x300x253.68	(850)	859	3922.87	2613.56
900x300x291.45	(900)	900	4940.65	3146.16

Section Name (PBP)	Label	Dept (mm)	I_{xx} (10^{-6} m^4)	M_p (kNm)
200x43.85	(200)	200	39.99	111.92
220x57.19	(220)	210	57.29	153.42
260x87.30	(260)	253	125.86	280.91
300x76.92	(300A)	299	160.06	296.60
300x184.11	(300B)	328	421.48	742.81
320x184.09	(320)	329	423.43	744.84
360x178.41	(360)	362	523.31	816.98
400x194.25	(400A)	364	577.59	897.02
400x212.52	(400B)	368	639.21	986.78
400x230.92	(400C)	372	702.55	1078.08

Table 3: Moment of inertia and nominal plastic moment capacity of some representative AISC I-sections.

Section Name	Depth d (mm)	I_{xx} ($10^{-6}\ m^4$)	M_p (kNm)
W 4 × 13	106	4.70	25.73
W 5 × 19	131	10.91	47.52
W 6 × 25	162	22.23	77.43
W 8 × 67	229	113.21	287.59
W 16 × 100	431	620.18	811.16
W 18 × 311	567	2896.97	3084.86
W 21 × 402	661	5078.02	4629.35
W 24 × 492	753	7950.02	6349.99
W 27 × 539	826	10613.90	7701.92
W 30 × 581	899	13735.64	9053.85
W 33 × 619	977	17398.47	10487.72
W 40 × 655	1108	23517.08	12536.10

6. Conclusion

Earthquake-resistant design of structures critically depends on the capacity design concept, wherein maximum moment capacity of members is expected to be mobilized under strong shaking. In this paper, the adequacy of Indian hot-rolled sections to resist strong earthquake effects has been examined. *Many* Indian sections do not meet the stability (or *compactness*) requirements specified in Indian standards as well as those of countries with advanced seismic provisions. Even those that satisfy the stability requirements, their sizes are so small that they are insufficient from strength and stiffness points of view to be able to construct large span and high rise earthquake-resistant constructions in strong seismic regions. For example, the nominal plastic moments of the deepest Indian section (ISWB600) with tapered and with parallel flange (NPB900) are only 988*kNm* and 3146*kNm*, respectively, in contrast to 12436*kNm* (W40655) in the deepest AISC section (1108*mm* deep).

Thus, although the technology is available in India to build steel MRF structures to resist strong seismic actions, the currently available Indian hot-rolled sections are inadequate to be used in large steel structures. Therefore, there is an urgent need to manufacture hot-rolled structural steel sections with higher plastic moment capacity. In the manufacture of the hot-rolled sections, tapered sections may be discontinued and parallel flange sections with higher plastic moment capacity may be developed. Until such time these sections become commonly available, the professional practice will require to design and construct based on built-up sections. However, built-up sections for use in severe seismic zones require special weld electrodes and processes; this aspect also requires to be developed in India. To facilitate building of tall structures, both these aspects, namely sections and welding technology require significant reconsideration. In closing, the Indian steel industry needs to research on both these aspects immediately.

References

AISC, (1989), Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design, American Institute of Steel Construction, Inc., USA, 1989.

AISC, (1994), *Metric Load and Resistant Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Illinois, USA, 1994.

AISC, (2002), *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Illinois, USA, 2002.

Arlekar, J. N., and Murty, C. V. R., (2002a), "P-V-M Interaction Curves for Seismic Design of Column Base Connections," Engineering Journal, AISC, 3rd Quarter, 2002.

Arlekar, J. N., and Murty, C. V. R., (2002b), "Improved Truss Model for Design of Welded Steel MRF Connections," Journal of Structural Engineering, ASCE, (in press).

Bresler. B., and Lin, T. Y., (1960), Design of Steel Structures, John Wiley & Sons, Inc., Publishers, USA, 1960.

Bruneau, M., Uang, C. M., and Whittaker, A., (1998), Ductile Design of Steel Structures, MaGraw-Hill Companies, Inc., NY, USA, 1998.

Engelhardt, M. D., and Sabol, T. A., (1998), "Reinforcing of Steel Moment Connections with Cover Plates: Benefits and Limitations," Engineering Structures, Vol.20, No.4-6, pp 510-520, 1998.

FEMA 267, (1995), "Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures," Report No.SAC-95-02, SAC Joint Venture, CA, USA, 1995.

Goswami, R., Arlekar, J. N., and Murty, C. V. R., (2003), "Concerns on Seismic Moment-

Shear Connections using available Indian Hot-Rolled I-Sections,”

IS 800, (1984), Indian Standard Code of Practice for General Construction in Steel, Bureau of Indian Standards, New Delhi, 1995.

IS 12778, (01.10.2003), Dimensions of Rolled Parallel Flange Beam Column and Bearing Pile Sections, Draft code for revision, Bureau of Indian Standards, New Delhi, 1995.

Malley, J. O., and Frank, K., (2000), “Materials and Fracture Investigations in the FEMA/SAC PHASE 2 Steel Project,” 12th World Conference on Earthquake Engineering, Paper ID. 2544, New Zealand, 2000.

Paul, S., Murty, C.V.R., and Jain, S.K., (1999), “Residual Stresses and Local Buckling in Indian Standard Hot-Rolled Steel Sections,” The Bridge and Structural Engineer, The Journal of ING-IABSE, Vol.29, No.4, pp 1-12, 1999.

Paul, S., Murty, C.V.R., and Jain, S.K., (2000), “Drift-based Re-sizing of Steel Frames Including Joint deformations,” The Bridge and Structural Engineer, The Journal of ING-IABSE, Vol.81, pp 91-100, December 2000.

Paul, S., Murty, C.V.R., and Jain, S.K., (2000), “State-of-the-art Review of Seismic Design of Steel Moment Resisting Frames – Part I: General Considerations and Stability Provisions,” Journal of Structural Engineering, Vol.27, No.1, pp 23-32, 2000.

Paulay, T., and Priestley, M.J.N., (1992), Seismic Design of Reinforced Concrete and Masonry Buildings, John Wiley and Sons, Inc., New York.

Penelis, G.G., and Kappos, A.J., (1997), Earthquake – Resistant Concrete Structures, E & FN Spon, Great Britain, 1997.

SP6(1), (1964), Indian Standard Handbook for Structural Engineers: Structural Steel Sections, Indian Standards Institution, New Delhi, 1964.

UBC, (1994), Uniform Building Code, 1994 Edition, International Conference of Building Officials, CA, USA, 1994.

Notations

The following symbols are used in this paper:

b	=	Width of plate element
b_f	=	Width of flange of section
d	=	Depth of member
d_w	=	Depth of web
E	=	Young’s modulus of steel

f_u	=	Ultimate nominal/characteristic stress
f_y	=	Minimum specified nominal/characteristic yield stress of steel
t	=	Thickness of plate element
t_f	=	Thickness of flange
t_w	=	Thickness of web
I	=	Moment of inertia of the section
M	=	Bending moment
M_p	=	Section plastic moment capacity using minimum specified yield
M_r	=	Maximum moment capacity of slender sections
P	=	Axial load
P_u	=	Factored axial load
P_y	=	Yield load
R	=	Section capacity modification factor
R_s	=	Strength reduction factor due to strain hardening of steel
R_y	=	Strength reduction factor due to uncertainty in the estimation of yield strength
ε	=	Normal strain
ε_r	=	Rupture strain
ε_{sh}	=	Strain-hardening strain
ε_u	=	Ultimate strain
ε_y	=	Yield strain
ϕ	=	Resistant safety factor
φ	=	Curvature
φ_y	=	Yield curvature
λ	=	Slenderness parameter
λ_p	=	Limiting slenderness parameter for compact section
λ_{pd}	=	Limiting slenderness parameter for compact section with minimum guaranteed plastic rotation capacity
λ_r	=	Limiting slenderness parameter for non-compact section
μ	=	Curvature ductility of the section
μ_y	=	Yield curvature ductility
μ_{sh}	=	Strain-hardening curvature ductility
μ_u	=	Ultimate curvature ductility
θ_p	=	Joint plastic rotation