

# Concerns on Seismic Moment-Shear Connections using available Indian Hot-Rolled I-Sections

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## Abstract

A rational capacity design method is presented for determining connection design forces considering increased yield stress of steel, strain-hardening of the beam /column cross-section, and local instability of the beam /column flange and web due to slenderness. A procedure is presented for arriving at shear-moment interaction boundaries at different axial load levels for commonly available Indian steel I-sections. The design procedure outlined in SP:6(6) needs to be upgraded and available Indian sections needs to be remolded to have larger flange widths.

## 1. Introduction

Satisfactory performance of steel structures during strong seismic shaking depends on numerous factors, including the three significant factors namely stability, strength and ductility of individual members. Apart from these, connections between members play an important role in the overall seismic performance of steel structures; inadequate connections can result in failure of structures even when structural members are adequately designed. A rational method for moment-shear connection design coupled with a preferred collapse mechanism is essential in achieving a ductile response of the whole structure during strong earthquake shaking. This paper presents one such connection design method and compares it with the current design method.

## 2. Connection Design Philosophy

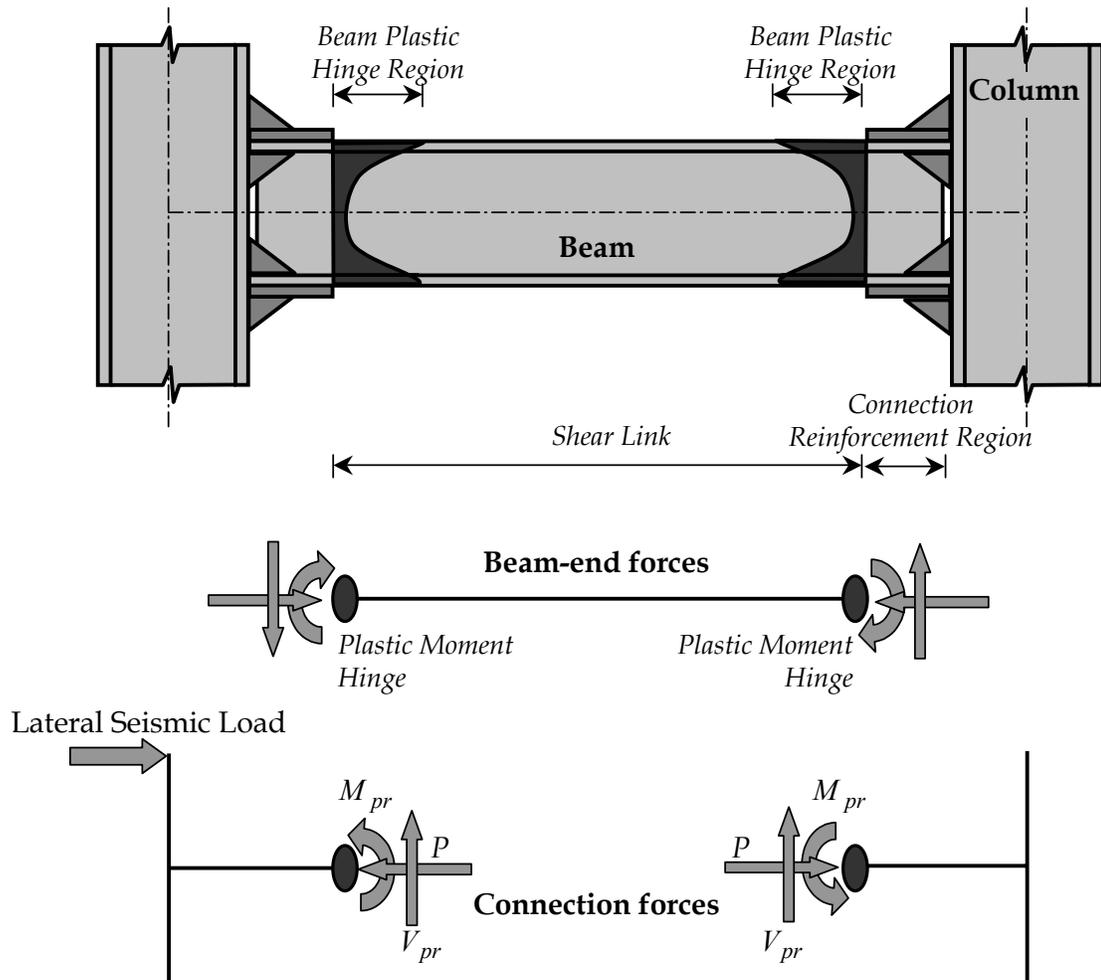
Following the large number of connection failures occurred during the 1994 Northridge earthquake (USA) and 1995 Kobe earthquake (Japan), a fresh approach emerged for the design of beam-to-column and column-to-base connections. Beam-to-column connections are designed now as per the *Capacity Design Concept*, discussed in an earlier paper [Goswami *et al.*, 2003]. By this design method, premature fracture of welds or fasteners is avoided at the connection. Here, beams are allowed to undergo ductile yielding, and connections are forced to remain elastic by designing them for the maximum demand arising from the members (beam, column) under plastic condition (Figures 1 and 2).

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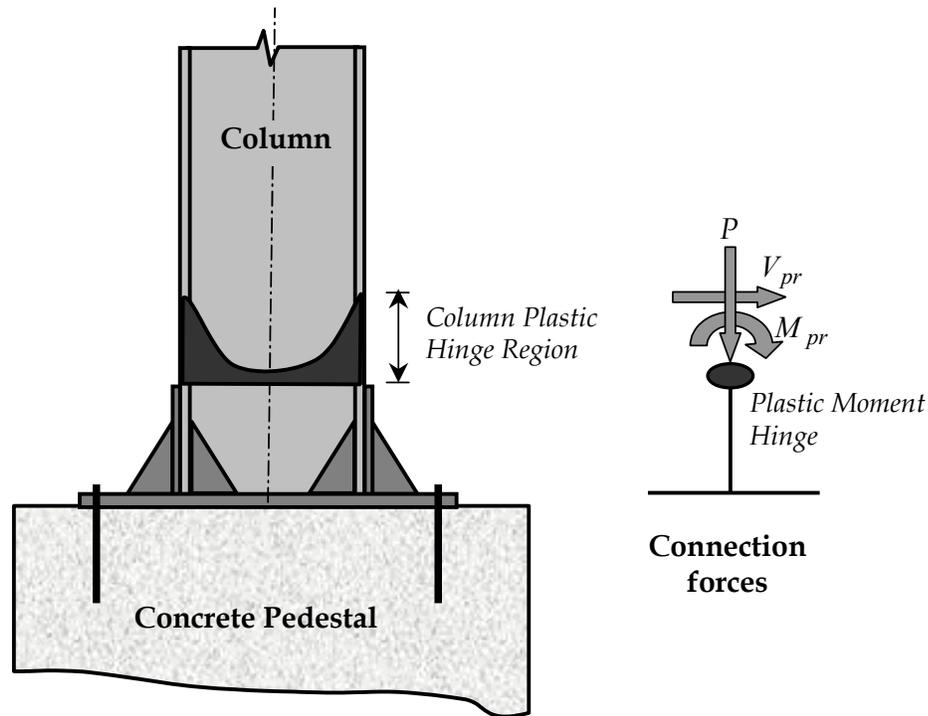
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**Figure 1:** Location of ductile plastic hinges adjacent to *beam-to-column* connections.

### 3. Connection Design Forces

The connection design forces generally accounts for the member nominal strength, *i.e.*, moment capacity  $M_{pr}$  and the associated equilibrium compatible shear  $V_{pr}$ , including effect of gravity load. Using  $M_{pr}$  in the connection design indirectly accounts for some strain-hardening of the beam and has the advantage of being simple to apply. However, to formalize the actual behaviour, a realistic stress-strain relationship for steel with strain-hardening must be used to assess the maximum demand imposed by the beam and column members on the connection elements.

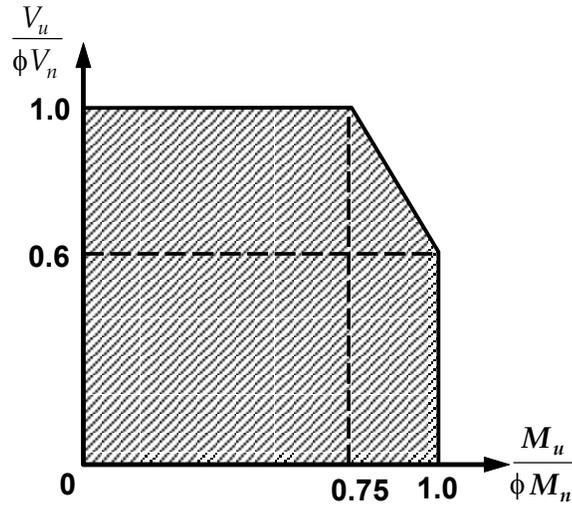


**Figure 2:** Location of ductile plastic hinges adjacent to *column-to-foundation* connections.

### 3.1 Axial Load - Shear - Moment ( $P$ - $V$ - $M$ ) Interaction

Under seismic action, all columns and beams of lower storeys in tall buildings under seismic actions are subjected to combined action of axial force, bending moment and shear force. To calculate the design forces on the connections, interaction between the axial force-shear-moment capacities of the members needs to be considered. In the shear-moment interaction for I-sections based on the maximum shear strength criterion for yielding [Hodge and Brooklyn, 1957], the yield strength  $f_y$  was assumed to be the limiting strength, and strain-hardening of steel was not considered. Approximate shear-moment ( $V$ - $M$ ) interaction curves proposed for deep beams (plate girders) were based on the tension-field action of the web [Basler, 1962]. The shear capacity of the web was reportedly not affected by the bending moment on the section so long the flanges did not yield. The web shear capacity dropped quickly as yielding of the beam flanges increased. Even this study does not consider the effect of strain-hardening in steel. Based on another study [Cooper *et al*, 1978], a linear interaction between shear and bending moment for plate girder design when the design shear was more than 60% of the factored nominal shear capacity and the bending moment was more than 75% of the factored nominal bending moment capacity of the section was given (Figure 3)

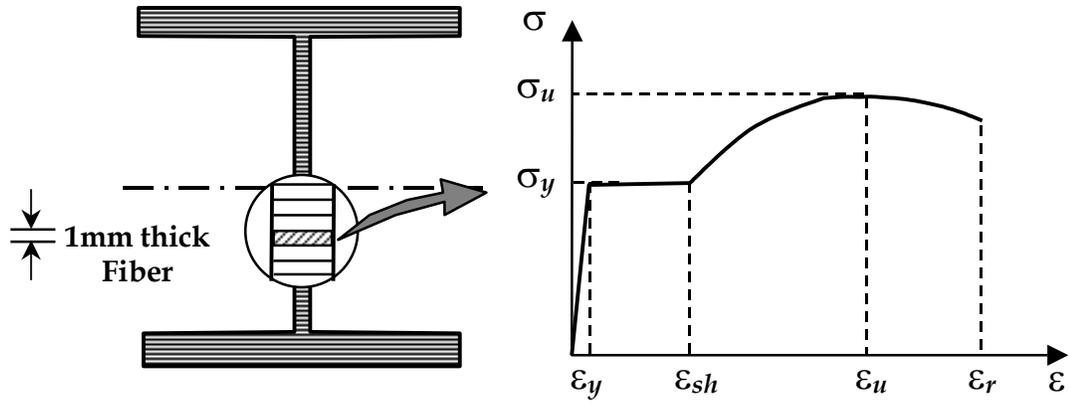
[AISC, 1994]. Such an interaction, however, is not considered in the IS code [IS 800, 1984].



**Figure 3:** AISC-LRFD shear-moment interaction. Shear-moment interaction is prescribed only for I-shaped plate girders with slender webs.

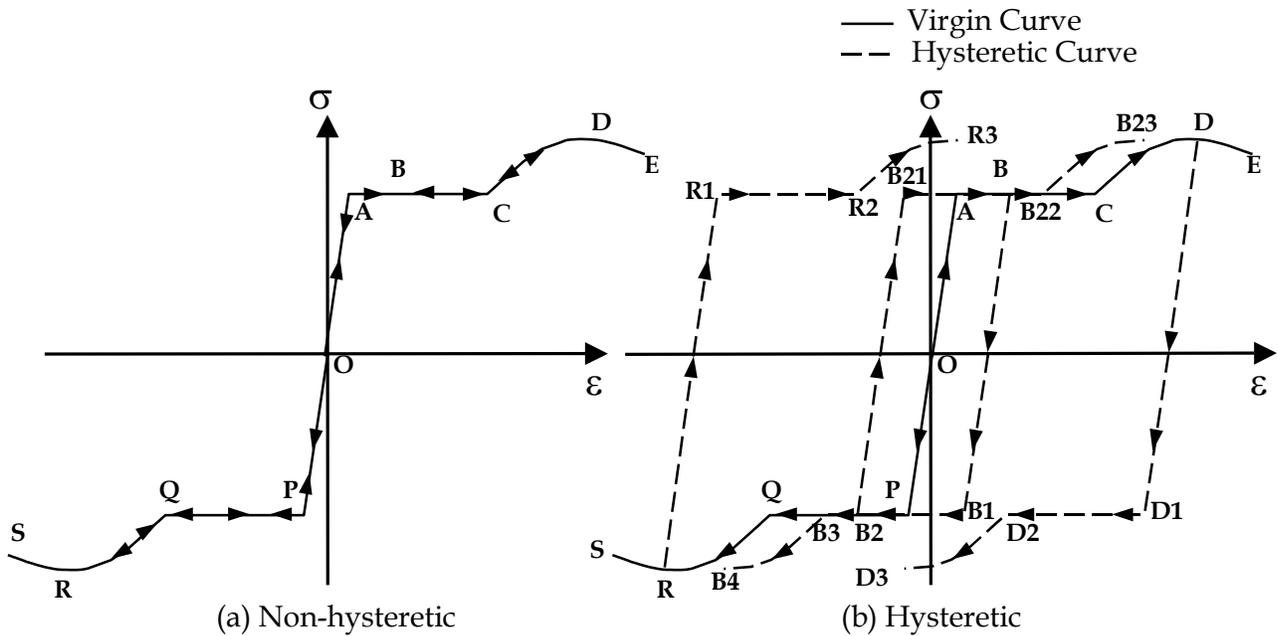
Like in the  $V$ - $M$  interaction, the existing axial force-moment ( $P$ - $M$ ) interaction curves for steel sections do not consider strain-hardening of steel in design. Moreover, the hysteretic behavior of the material is also not considered. Thus, while obtaining the moment-curvature ( $M$ - $\phi$ ) curves, the strain profile resulting from the simultaneous application of axial load ( $P$ ) and a specific curvature  $\phi$  is imposed on the section in one step starting with zero initial curvature and zero initial axial strain, irrespective of the state of the section at the immediately preceding curvature value; the stresses in the fibers are obtained directly from the virgin stress-strain curve.

In this study, a fiber model [Murty and Hall, 1994] (Figure 4) is used to develop the  $P$ - $V$ - $M$  interaction curves for sections subjected to known compressive axial loads. Due to the presence of the axial load, the section is already subjected to some initial axial strain. Now, if this section is subjected to a specific curvature  $\phi$ , to keep the axial load  $P$  constant, the axial strain in the section also changes if the section goes into inelasticity. A strain-hardened stress-strain curve of steel with the rules for hysteretic behavior is used in this study (Figure 5). A stressed fiber returns along the virgin stress-strain curve only within the initial elastic range. Fibers that are subjected to increased axial strain will continue along the virgin stress-strain curve, and those subjected to reduced strain will return along (a) the virgin stress-strain curve if the fiber is in elastic



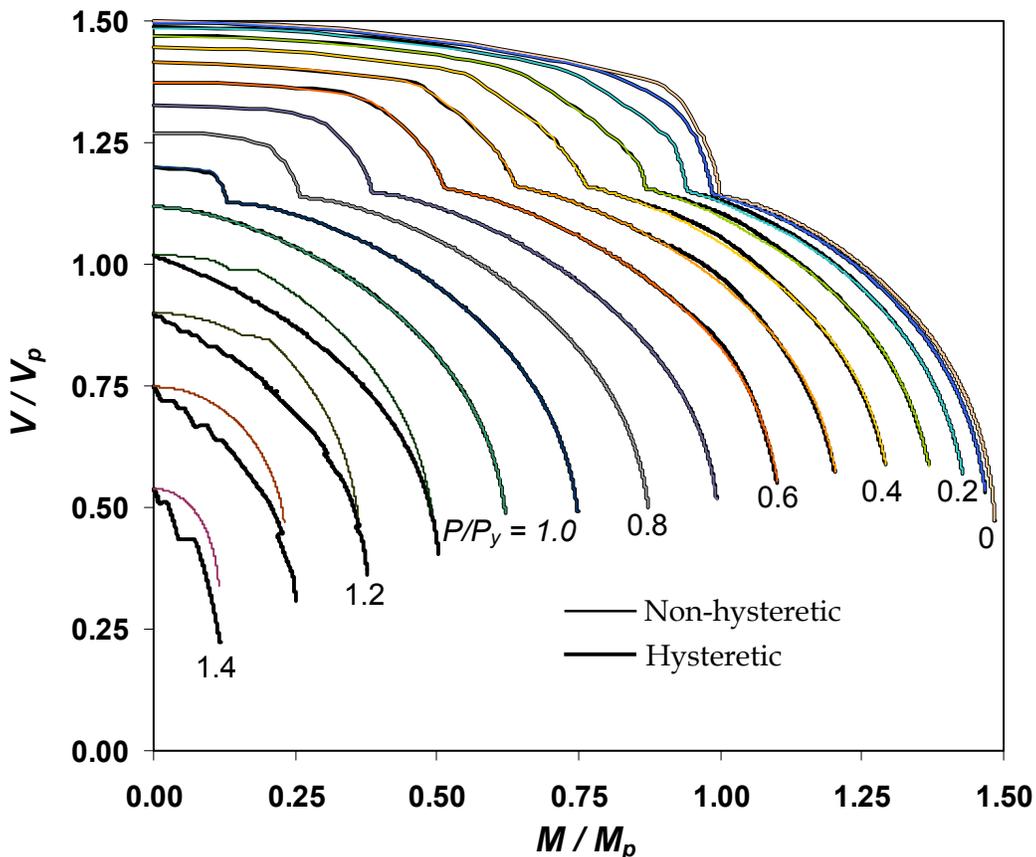
**Figure 4:** *Fiber model* showing the discretization of the beam section along with the explicit form of stress-strain relationship for steel [Murty and Hall, 1994] used in this study.

range, or (b) the new unloading stress-strain curve, which is parallel to the initial elastic portion of the virgin stress-strain curve if the fiber is in the inelastic range. Thus, for fibers already beyond the elastic limit, unloading takes place along a new unloading curve. On further unloading, some fibers may reach the translated virgin stress-strain curve in the other direction, and from then on they follow the same path [Arlekar and Murty, 2002].

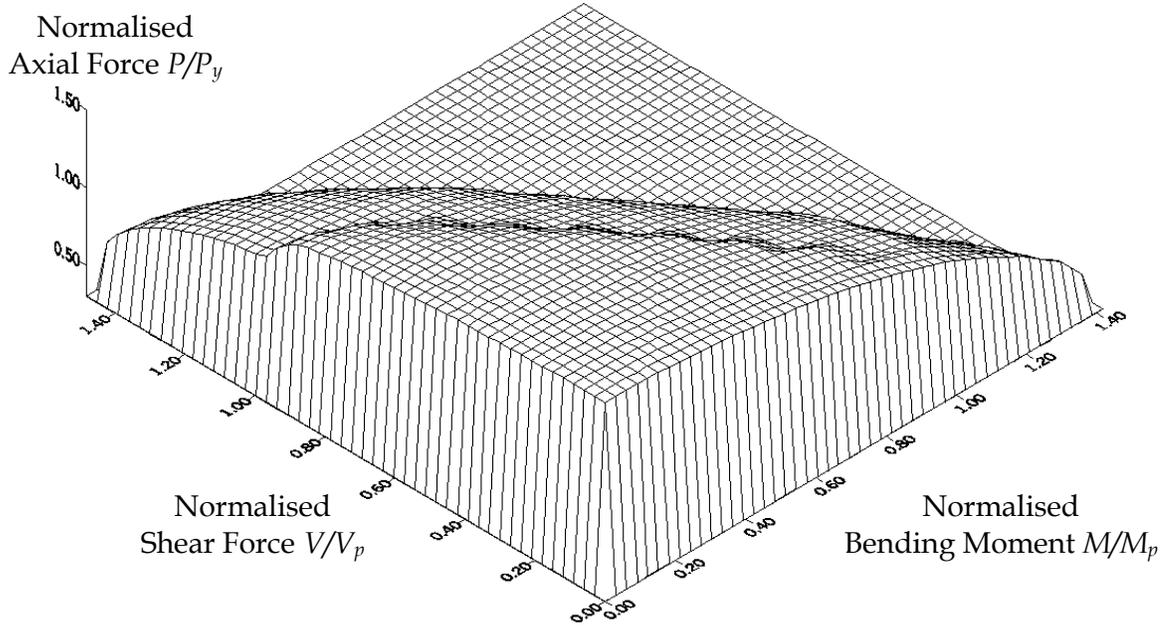


**Figure 5:** Schematic representation of the loading and unloading paths for steel.

Normalized  $V$ - $M$  interaction curves for typical ISMB 600 section [SP:6(1), 1964] for various levels of the compressive axial load are obtained as discussed above (Figure 6). The moment is normalized with the nominal plastic moment capacity  $M_p (= f_y Z)$  and shear with the nominal shear capacity  $V_p (= \tau_y t_w d)$ . The  $V$ - $M$  interaction curves obtained using a strain-hardened *virgin* stress-strain curve are also shown in Figure 6. The  $V$ - $M$  curves without hysteretic loading are marginally higher than the corresponding curves obtained using the hysteretic loading, only when the axial load is higher than the yield load  $P_y$  (Figure 6). Thus, the  $V$ - $M$  curves with non-hysteretic loading, commonly used in codes, are acceptable in static design where the axial load does not change or is below the member yield load. However, under earthquake shaking, the axial load can swing by large amount and the  $V$ - $M$  curves with hysteretic loading better reflect the actual lower member capacity and should be considered for the *member* design. Figure 7 shows the three-dimensional hysteretic  $P$ - $V$ - $M$  interaction surface for ISMB 600. It is, however, noteworthy that the use of non-hysteretic curves for connection demand estimate results in a conservative approach.



**Figure 6:** *Shear-Moment interaction* showing normalized  $V$ - $M$  curves for a typical ISMB 600 for different axial load levels with and without hysteretic stress-strain curve.



**Figure 7:** Strength interaction in ISMB 600: Normalized  $P$ - $V$ - $M$  interaction surface of ISMB 600 generated using hysteretic stress-strain model.

While developing the  $P$ - $V$ - $M$  interaction curves, the following is considered. For a given normal stress  $\sigma_{xx}$  (due to axial load and bending moment) in a fiber, the von-Mises yield criterion for steel represented by

$$\sigma_{xx}^2 + 3\tau_{xz}^2 = Y^2, \quad (1)$$

is used to calculate the available shear capacity  $\tau_{xz}$ . Here,  $Y$  is taken as the ultimate stress  $f_u$ . The curvature is increased from zero to a maximum value corresponding to the maximum strain  $\varepsilon_r$  at the extreme fiber, and at each level, the shear and normal capacities are estimated. The uniaxial stress-strain curve of steel (Figure 5) has a drop in the stress beyond the strain  $\varepsilon_u$  corresponding to the ultimate stress. The limiting shear stress from Eq. (1) when  $\sigma_{xx} = f_u$ , is zero. For strains greater than  $\varepsilon_u$ , Eq. (1) suggests that the shear stress  $\tau_{xz}$  in fibers is non-zero. However, in this study it is assumed that all fibers having strains beyond  $\varepsilon_u$  do not have shear capacity. Further, while obtaining the limiting  $V$ - $M$  boundary, it is assumed that beam flanges and webs do not undergo buckling. The nominal shear strength ( $V_p = \tau_y t_w d$ ) and the nominal bending moment capacity ( $M_p = f_y Z$ ) of the section are used to normalize the shear and moment capacities, respectively. The first yield shear stress corresponding to a state of pure

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shear is used and defined as  $\tau_y = f_y / \sqrt{3}$ .

### 3.2 Axial Load - Moment (P-M) Interaction at Zero Shear Force

The  $P$ - $M$  interactions obtained in this study, using the fiber model with hysteretic stress-strain curve for hot-rolled Indian I-sections are shown in Figure 8. An upper bound of the normalized  $P$ - $M$  interaction curves for zero shear can be expressed by the following expression

$$\frac{M}{M_p} = \frac{f_u}{f_y} \left( 1 - \frac{f_y}{f_u} \frac{P}{P_y} \right)^{1.54} . \quad (2)$$

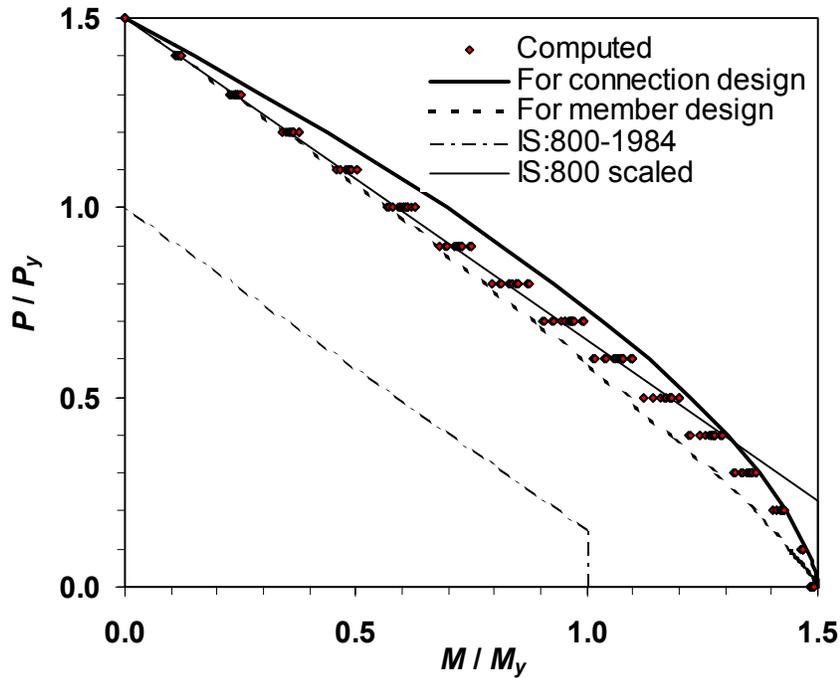
This upper bound limit is conservative for ascertaining the connection demand forces. Using this for member design would result in an overestimate of the member capacity and thus, a lower bond limit is required. This is also shown in dotted line in Figure 8. The design codes generally give such lower bound limit for member design purpose. However, using this lower bound limit for connection design would result in an underestimate of the maximum demand on the connection components and may lead to premature failure of the connection before the member capacity is reached.

The Indian Standard IS:800-1984 assumes a bilinear  $P$ - $M$  interaction curve as also shown in Figure 8. Since, the normalized  $P$ - $V$ - $M$  curves obtained in this study are for fully strain-hardened condition, the IS interaction curved is scaled to the ultimate strength capacity  $f_u$ . The IS curve depicts an average member capacity for moments up to about the nominal plastic moment  $M_p$ . Thus, it underestimates the moment capacity of some members, and hence, connections designed using this interaction would be under-designed.

## 4. Section Capacity Modification Factors

The  $P$ - $V$ - $M$  curves developed in this study are for the full capacity of the section without considering the effect of uncertainty in the estimation of yield strength, compactness of the section, slenderness of the member, and the stability against flexural-torsional buckling of the member. The first factor mentioned above is related to the strength of the member, and the latter three are related to the stability of the member. Taking into account all the section capacity modification factors, the connection design moment is then given by

$$M_{pr} = M_p R_y R_s R_c . \quad (3)$$



**Figure 8:**  $P$ - $M$  interaction curve along with the actual  $P$ - $M$  points for  $V = 0$  for Indian Hot Rolled I-Sections.

Considering the plastic hinges at the ends of the beam with moments  $M_{pr}$ , the corresponding equilibrium compatible shear design force  $V_{pr}$  on the connections is then determined. These, together with the design axial load as appropriate, are the total demand force on the connections. The effects of these factors and the method of incorporating them in the member capacity obtained from the  $P$ - $V$ - $M$  curves developed in this study are as follows.

#### 4.1 Yield Strength of Material

The existing code procedures for the design of members are based on the minimum specified yield strength  $f_y$  of the steel. The uncertainty in material strength can cause overstrength and this should be accounted [Goswami *et al.*, 2003]. AISC-SPSSB provisions recommend the use of higher yield strength while calculating the member strength for the determination of the design forces for connection elements [AISC, 2002]; the ratio  $R_y$  of the expected yield strength to the minimum specified yield strength of the connected member as suggested by AISC [AISC, 2002] varies from 1.1 to 1.3 for different grades of steel. In absence of such data of  $R_y$  for the Indian sections, a value of 1.0 is used for the  $P$ - $V$ - $M$  curves obtained previously, using a yield strength of 250MPa.

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## 4.2 Strain Hardening of Steel

Strain-hardening of steel cause increase in member capacity, and hence demand on the connections. Thus, a strain-hardening factor  $R_s$  is introduced given by the following [Goswami *et al.*, 2003]:

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

where  $\mu$  is the curvature ductility imposed on the section. For steel of  $f_y = 250\text{MPa}$  and the Indian sections, the value of  $R_s$  is in the range 1.0 to 1.24. The  $P$ - $V$ - $M$  curves developed in this study are based on strain-hardening stress-strain curve for steel. Thus, the use of these curves for calculating the maximum member capacities includes the effect of strain-hardening.

## 4.3 Compactness of the Section

Local buckling of flanges and web of the column adversely affect its maximum strength. Since the column capacity,  $M$ , as determined from the  $P$ - $V$ - $M$  interaction does not consider the effect of the compactness of the section, a compactness factor  $R_c$ , is introduced to account for the reduction in the maximum achievable member capacity owing to premature local buckling given by

$$R_c = \begin{cases} 1.0 & \text{for } \frac{b}{t} \leq \lambda_p \\ 1.0 - 0.2 \left\{ \frac{(b/t) - \lambda_p}{\lambda_r - \lambda_p} \right\} & \text{for } \lambda_p < \frac{b}{t} \leq \lambda_r \\ 0.8 & \text{for } \frac{b}{t} > \lambda_r \end{cases} \quad (5)$$

Here, the minimum value of  $R_c$  is 0.8. The limiting values for  $\lambda_r$ ,  $\lambda_p$  and  $\lambda_{pd}$  are prescribed in the codes. However, the limits prescribed in the codes are originally for the purpose of beam design and thus, will tend to give a conservative underestimate of the member strength. But, in connection design, the upper bound strength is required. Moreover, these values are for prismatic flange and web. For Indian hot rolled I-sections with tapered flanges, such limits of  $\lambda_r$ ,  $\lambda_p$  and  $\lambda_{pd}$  for connection design purpose and dependence of  $R_c$  on these needs to be prescribed.

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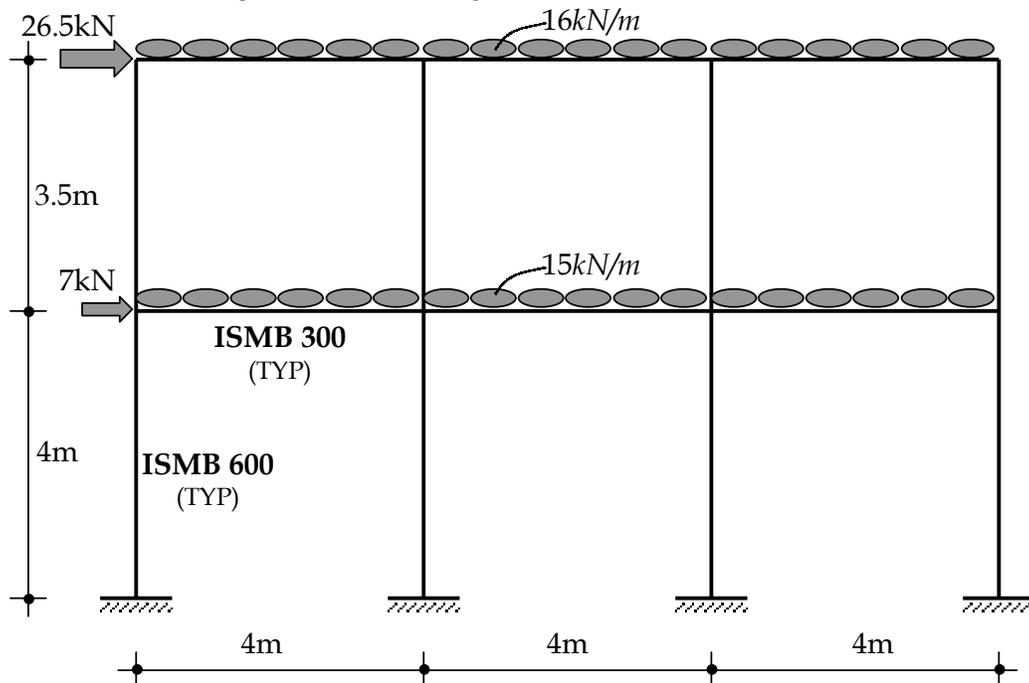
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## 5. Connection Design Examples

To illustrate the current Indian connection design practice and its limitations, consider a MRF with ISMB 300 beam and ISMB 600 columns with spans as shown in Figure 9. The bay span considered is  $4\text{m}$ . The uniformly distributed gravity load (including dead load, fraction of live load on roof and floor as per IS 1893 (Part I), 2002, and roof finish load) is  $16\text{kN/m}$  on the roof beam and  $15\text{kN/m}$  on the floor beam. Considering the structure to be in seismic zone V [IS 1893 (Part I), 2002], it is analysed for the different load combinations. The maximum joint moment and shear forces for load combination  $1.3(\text{DL}+\text{LL}+\text{EL})$  are respectively  $43\text{kNm}$  and  $48\text{kN}$ . The beam-to-column connection is designed for this force.

### 5.1 Common Design Practice

In the common design practice, members and connections are designed based on the linear static analysis results. The web is considered to carry the shear and the two flanges carry the flexure, in the form of tension and compression. Accordingly, from the static analysis results above,  $6\text{mm}$  fillet welds of  $100\text{mm}$  length on both sides of the web are sufficient to carry the shear. The flanges can be connected to the column through a  $10\text{mm}$  full penetration butt weld, or by  $10\text{mm}$  fillet weld along the straight portions of the flanges. As there are no particular recommendations for the type of connection arrangement to be adopted in the existing Indian Standards, such simple form of connection can be designed still adhering to the code provisions, if desired.



**Figure 9:** Structural arrangement: Member sizes, boundary conditions with loadings.

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## 5.2 Design Procedure in SP:6(6)

As a second step, the same beam-to-column connection is checked following the design philosophy given in the ISI Handbook for Structural Engineers: Application of Plastic Theory in Design of Steel Structures [SP:6(6), 1973]. SP:6(6) recommends that the connections be designed for the nominal plastic moment that is to be transmitted from one member to another. Thus, under the condition of an extreme shaking, assuming that plastic hinges are formed at the beam ends, the design forces for connection design is the nominal plastic moment  $M_p$  (161.6kNm) of the beam and a shear of 120.6kN, considering the critical sections to lie at the beam-column interface. Thus, now the design shear and moment are increased by 1.5 to 2.8 times over the structural analysis results of the frame discussed earlier, and now, the connection designed earlier becomes inadequate. Further, the code does not specifically say that plastic analysis and design needs to be done for seismic conditions. Thus, it remains at the hand of the designer to choose the type of analysis and design one wishes to do, and in the process, the structural safety is put at stake.

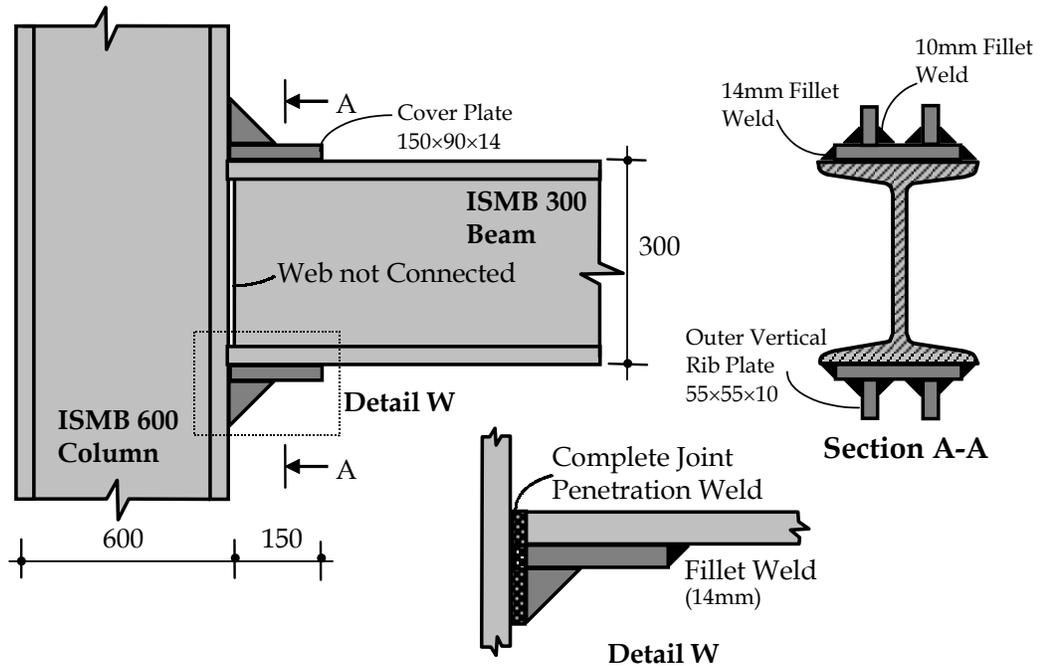
## 5.3 A Rational Design Procedure

Supplementary to the above, as discussed previously, now considering the overstrength factors  $R_y = 1.3$ ,  $R_s = 1.24$  and  $R_c = 1.0$ , the design moment at the column face becomes 261kNm plus the shear times the length of the connection reinforcement region and plastic hinge length. Thus, there is an increase of more than 62% in the design moment alone. This can cause premature failure of the connection even before the beam reaches its full plastic capacity resulting in collapse of the structure. In addition, such simple connection schemes discussed in Sections 5.1 and 5.2 do not facilitate smooth flow of forces through the connection region; stress concentration at the beam flange-column flange junction severely affects the functioning of the connection. To address these difficulties, a rational method of design of moment-shear connections is developed wherein a cover plated rib plated connection is designed for the overstrength beam forces [Arlekar and Murty, 2003]. A step-wise procedure presented for standard AISC sections, is extended here for the current design problem with Indian sections. Following this, the associated shear force also increases to 186.5kN, an increase of about 55% compared to a value of 120.6kN in Section 5.2. The resulting connection configuration is shown in Figure 10. However, this scheme works best on sections with wide non-tapered flanges; Indian sections have very small flange

width [Goswami *et al.*, 2003]. As such, the available width of cover plate is much less and may be insufficient to transfer the forces in higher structures with higher forces; the example frame is a nominal two-storey lightly loaded structure chosen only with the intention to discuss the important issues. Also, due to tapering of these flanges, additional inner rib plates cannot be provided efficiently to further reinforce the connection, if required.

### 5.4 Discussion

In the above, a rational method for moment-shear connection design is proposed. Moreover, the design procedure outlined in SP:6(6) is found to be inadequate in that it does not account for the increase in the maximum demand that may be mobilized due to the overstrength factors discussed in Section 4. Also, given the sizes of the available hot-rolled section, it may not be possible to develop moment-shear connections for tall structures in high seismic areas.



**Figure 10:** Beam-to-column connection arrangement: Geometry, location of connection elements and type of welds.

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## 6. Conclusion

In the recent times, the design of connections in welded steel MRFs has seen a major change. Most developed codes now recommend that the connections for MRF should be designed using the capacity design concept. This means that the connections should be able to resist and transfer the forces and deformations corresponding to the maximum capacity that is expected to be mobilized in the connected members. In this regard, axial load-moment-shear interaction plays an important role on deciding upon the maximum mobilized demand. Further, material strain hardening and higher material strength over the nominal specified values significantly increases the demand on the connections over the code specified values. With this, although the basic perspective of moment-shear connection design is in light, the idea can be effectively put to practice only with clearly laid out code provisions and availability of appropriate raw materials, namely proper wide-flange non-tapered hot-rolled sections.

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## Notations

*The following symbols are used in this paper:*

$b$	=	Width of plate element; width of flange of section
$d$	=	Depth of member
$f$		Stress
$f_u$	=	Ultimate normal stress
$f_y$	=	Minimum specified normal yield stress of steel
$t$	=	Thickness of plate element
$M$	=	Bending moment
$M_n$	=	Nominal flexural strength of member
$M_p$	=	Section plastic moment capacity using minimum specified yield
$M_{pr}$	=	Connection design moment
$M_u$	=	Factored moment
$P$	=	Axial load
$P_y$	=	Yield load
$R$	=	Section capacity modification factor
$R_c$	=	Strength reduction factor due to compactness
$R_s$	=	Strength reduction factor due to strain hardening of steel
$R_y$	=	Strength reduction factor due to uncertainty in the estimation of yield strength
$V$	=	Shear force
$V_n$	=	Nominal shear capacity of section
$V_p$	=	Section plastic shear capacity using minimum specified yield
$V_{pr}$	=	Connection design shear

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$V_u$	=	Factored shear strength of member
$Y$	=	Failure stress in Von Mises criterion
$Z$	=	Plastic section modulus of the member
$\varepsilon$	=	Normal strain
$\varepsilon_r$	=	Rupture strain
$\varepsilon_{sh}$	=	Strain-hardening strain
$\varepsilon_u$	=	Strain corresponding to ultimate stress
$\varepsilon_y$	=	Yield strain
$\phi$	=	Resistant safety factor
$\varphi$	=	Curvature
$\lambda$	=	Slenderness parameter
$\lambda_p$	=	Limiting slenderness parameter for compact section
$\lambda_{pd}$	=	Limiting slenderness parameter for compact section with minimum guaranteed plastic rotation capacity
$\lambda_r$	=	Limiting slenderness parameter for non-compact section
$\mu$	=	Curvature ductility of the section
$\mu_y$	=	Yield curvature ductility
$\mu_{sh}$	=	Strain-hardening curvature ductility
$\mu_u$	=	Ultimate curvature ductility
$\sigma, \sigma_{xx}$	=	Normal stress
$\sigma_y$	=	Yield stress
$\sigma_u$	=	Ultimate stress
$\tau_{xz}$	=	Shear stress
$\tau_y$	=	Minimum specified shear yield stress of steel