Journal of Structural Engineering
Vol.27 No.2 July 2000 pp.117-124

No.27–12

# State-of-the-art review of seismic design of steel moment resisting frames Part II[+]: Strength and Drift criteria

Sudip Paul*, C.V.R. Murty** and Sudhir K. Jain***

**This part of the paper presents a comprehensive state-of-the-art review of seismic design provisions related to strength and drift criteria as given in American and Indian codes. The necessity of the different strength provisions in various design methods available in the American codes is discussed. A discussion on the stability provisions has already been presented in Part I of this paper.[1] This part draws a parallel of the available Indian provisions with those of the American codes, in respect of strength and drift criteria to study their range of applicability.**

The design seismic force is usually a fraction of the actual force experienced by the structure during strong earthquake shaking. For this reason structures in high seismic regions tend to undergo significant inelastic action. Fortunately, steel as a material provides a very stable post-yield behaviour and hence, steel structures demonstrate good hysteretic behaviour. However, the post-yield deformation of the structure is dependent on the level of the design seismic force. If the design force is small, the post-yield deformation is large, thereby placing an unduly high demand on the ductility of the structures. Thus the design specifications on the strength, stiffness and ductility are inter linked. The ductility provisions in the current seismic codes are discussed in Part I of this paper[1]. This part discusses the strength and drift (stiffness) provisions associated with the extent of ductility achievable through the stability provisions available in the current seismic codes.

## DESIGN CRITERIA

The design criteria for steel structures can be broadly categorised into (a) the strength criteria, and (b) the drift criteria. Though the strength criteria form the basis for preliminary design, in medium to high rise steel moment resisting framed structures, selection of the final design sections may also be governed by the drift criteria.

### Regular Load Combinations

The American Institute of Steel Construction (AISC) design methods: Allowable Stress Design (ASD)[2], Plastic Design (PD)[2] and Load and Resistance Factor Design (LRFD)[3], and National Earthquake Hazard Reduction Program (NEHRP)[4] recommendations do not specify any design load combinations. Instead, they refer the designer to follow the Loads and load combinations as per the code under which the structure is designed (e.g., Uniform Building Code (UBC)[5]). However, the NEHRP provisions give different values of seismic force for load combinations depending on whether the effects of gravity and seismic loads are additive or counterative. In the following methods even those load combinations which do not involve seismic force are covered. This is because in a low seismic zone, the design of components may be governed by the non-seismic combination and the transition from non-seismic to seismic will also depend on load factors for non-seismic combinations.

*ASD Approach:* The UBC 1997 provides two sets of load combinations when using ASD approach and the designer is allowed to choose appropriate set for design. The first set provides the following combinations:

$$D$$
$$D + L + (L_r \text{ or } S) \qquad (1)$$

---

+ Part I of this paper was published in Vol.27, No.1 of this Journal

* Engineer, ETD(A), Engineers India Limited, New Delhi - 110 066, India (Formerly Graduate Student, Dept. of Civil Engg., IIT Kanpur)
** Associate Professor, *** Professor } Department of Civil Engineering, Indian Institute of Technology, Kanpur - 208 016, India

$$D + (W \text{ or } E/1.4) \quad (1)$$

$$0.9D \pm E/1.4$$

$$D + 0.75\ [L + (L_r \text{ or } S) + (W \text{ or } E/1.4)]$$

while using this load combination no increase in permissible stresses as permitted. However while using the following load combinations, namely

$$\begin{aligned} &D + L + (W \text{ or } E/1.4) \\ &D + L + W + S/2 \\ &D + L + S + W/2, \text{ and} \\ &D + L + S + E/1.4 \end{aligned} \quad (2)$$

UBC 1997 permits a 33.3% increase in permissible stresses for those load combinations which include $W$ or $E$.

Seismic Provisions for Structural Steel Buildings (SPSSB)[6] specifications for load combinations are the same as those followed in LRFD approach but an increase of allowable stress by 70% is permissible.

The load combinations in IS: 800-1984[7], are as follows. An increase in the allowable stresses by 33.3% is allowed when earthquake or wind force are covered.

$$\begin{aligned} &D \\ &D + L \\ &D + E \text{ or } W, \text{ and} \\ &D + f_1 L \pm E \text{ or } W \end{aligned} \quad (3)$$

In the above equations $D$ represents dead load; $W$ is the wind load, $S$ is the snow load, and $E$ is the earthquake load. $L$ is the live load except on roof and $L_r$ is the roof live load. The factor $f_1$ is equal to 0.25 for live load class of 3.0 $kN/m^2$ or less, and is 0.5 for a higher live load class.

*PD Approach:* The load combinations as per IS-PD[8] method are

$$\begin{aligned} &1.7D \\ &1.7D + 1.7L \\ &1.7D + 1.7(W \text{ or } E) \\ &1.3D + 1.3L + 1.3\ (W \text{ or } E) \end{aligned} \quad (4)$$

*LRFD Approach*: Specifications for load combinations under UBC 1997 and SPSSB 1997 using LRFD approach refer to the AISC-LRFD specification. These are given by:

$$\begin{aligned} &1.4D \\ &1.2D + 1.6L + 0.5\ (L_r \text{ or } S) \\ &1.2D + 1.6(L_r \text{ or } S) + (f_1 L \text{ or } 0.8W) \\ &1.2D + 1.3W + f_1 L + 0.5(L_r \text{ or } S) \end{aligned} \quad (5)$$

where $f_1$ is defined under Eq.(3), and $f_2$ is the snow load multiplication factor and is equal to 0.7 for roof configurations that do not shed snow off the structure and 0.2 for others.

## SPECIAL LOAD COMBINATIONS

In addition to the above load combinations, UBC, SPSSB and NEHRP specify additional special load combinations involving seismic load for axial design for columns and design of connections. These load combinations are specified in conjunction with LRFD approach. However, the case of allowable stress design approach while using the load combinations, the stresses can be increased by 70%, provided no increase of 33.3% have been made as recommended in other clauses for combination of earthquake forces.

The UBC 1997 specifications are:

$$\begin{aligned} &1.2D + f_1 L + 1.0E_m, \text{ and} \\ &0.9D \pm 1.0E_m \end{aligned} \quad (6)$$

where $f_1$ the live load multiplication factor, 1.0 for live load classes in excess of 4.79 $kN/m^2$ and 0.5 for lower live load classes. $E_m$, the estimated maximum earthquake force, is given by $\Omega_0 E_h$, where $\Omega_0$ is the overstrength factor (2.8 or 2.2 depending on the type of the structural system) and $E_h$ is the horizontal earthquake force.

As per the SPSSB 1997 specifications, the special load combinations are:

$$\begin{aligned} &1.2D + 0.5L + 0.25 + E_m \\ &0.9D - E_m \end{aligned} \quad (7)$$

which are identical to those specified by UBC 1997. NEHRP 1997 also gives the expressions for amplified earthquake forces. It does not specify any load combinations.

## STRENGTH CRITERIA

### Column Requirements

*AISC-ASD and IS-ASD Specifications.* The elastic critical stress $F_{cr}$ under axial compression as per AISC-ASD specifications is given by:

$$F_{cr} = \pi^2 E/(kL/r)^2 \quad (8)$$

where $E$ is the modulus of elasticity, $L$ is the unsupported length of member, $k$ is the effective length factor indicating influence of boundary conditions, $r$ is the radius of gyration. The allowable stress $F_{allow}$ under axial compression is $12/23F_{cr}$, considering a factor of safety of 23/12.

In the presence of residual stresses, Eq.(8) which is based on elastic consideration is not valid for critical stress $F_{cr}$ more than $0.5F_y$. This is because some portions of the section may reach the yield stress even at a strain as low as $0.5\varepsilon_y$. Taking this as the limit of the elastic behaviour, i.e., for $F_{cr} = 0.5F_y$ and $F_y = 250$ $N/mm^2$, $(kL/r)$ is 126, this is called *critical elastic slenderness coefficient*, $C_c$ For slenderness ratios $(kL/r)$ less than 126, the column member is expected to undergo inelastic buckling and its critical stress is given by:

$$F_{cr} = F_y \left[1 - (kL/r)^2/2C_c^2\right] \quad (9)$$

The corresponding allowable stress is obtained by dividing the critical elastic stress with a factor of safety $F_s$ as given by:

$$FS = \frac{5}{3} + \frac{3}{8}\left(\frac{kL/r}{C_c}\right) - \frac{1}{8}\left(\frac{kL/r}{C_c}\right)^3 \quad (10)$$

which has a value of 23/12 when $(kL/r) = C_c$, and 1.67 when $(kL/r) \approx 0$.

The IS-ASD specification for permissible stress $\sigma_{ac}$ under axial compression is given by:

$$\sigma_{ac} = 0.6 f_{cc} \; F_y \Big/ \left[(f_{cc})^n + (F_y)^n\right]^{1/n} \quad (11)$$

where $n$ is assumed as 1.4 and $f_{cc}$, the elastic critical stress in compression is given by $\pi^2 E/\lambda^2$, but not more than $0.6F_y$.

*AISC-PD and IS-PD Specifications:* The AISC-PD approach limits the maximum slenderness ratio to 126, and thus, uses Eq.(11) to specify allowable stress. The IS-PD approach uses, an allowable stress value of 1.7 times of that specified in Eq.(11).

*AISC-LRFD Provisions:* The AISC-LRFD specifications require that the factored axial load does not exceed the following:

$$\begin{cases} \phi_c \; 0.877\pi^2 E/(kL/r)^2 & \text{for } (kL/r) > 133 \\ \phi_c \; (0.658\lambda_c^2) & \text{for } (kL/r) < 133 \end{cases} \quad (12)$$

where $\phi_c$ is the strength reduction factor (0.85); and the factor 0.877 accounts for the initial curvature; and $\lambda_c$ is the dimensionless LRFD slenderness parameter given by $\lambda_c^2 = (kL/r)^2 \; F_y/\pi^2 D$.

*AISC-SPSSB and UBC Special Seismic Provisions*: In addition to the above design criteria, some special requirements are available particularly for the seismic design of columns. As per the SPSSB specifications, if the design axial load exceeds 40% of the nominal column strength $\phi_c P_y$, the column should be designed both for axial compression and axial tension using load combinations given in Eqs.(7). However, the design load in any case may not exceed the minimum of (a) the loads that can be transferred to the column given by, $1.1R_y$ times the design strength of the connecting beams, where $R_y = 1.5$ for *Fe*-250 grade steel, and (b) the loads determined by the foundation capacity to resist overturning effect. As per the UBC 1997 specifications, if the axial compressive stress exceeds $0.3F_y$, the column axial strength should be checked for the load combinations of Eq.(6).

The welds in column flange splices are very brittle under tensile loading. Hence, the SPSSB specifications permit no column splices within 1.2m from the beam-column joint or half the clear storey height, whichever is smaller. Moreover, in designing the column splices, the design tensile forces should be increased by 50%. The welds in beam-column splices should be such that at least $0.5R_y$ of the yield strength of the smaller column flange area is developed, where $R_y$ is the same as above.

**Beam Requirements**

*AISC-ASD and IS-ASD Provisions:* AISC-ASD and IS-ASD limit the maximum allowable stress $F_{allow}$ in bending compression for compact sections as:

$$F_{allow} = 0.66F_y \quad (13)$$

For non-compact sections the AISC-ASD specifications for allowable stress is given by.

$$F_{allow} = F_y \left[0.79 - 0.00076 \; \frac{b_f}{2t_f}\sqrt{\frac{F_y}{k_c}}\right] \quad (14)$$

where $b_f$ and $t_f$ are the flange width and thickness of symmetric sections respectively and $k_c$ is a factor corresponding to the slenderness ratio of web and is given as $k_c = \dfrac{4.05}{(d_w/t_w)^{0.46}}$ for $d_w/t_w \geq 70$ else $k_c = 1$. The above expressions do not consider the case of lateral-torsional buckling.

For compact and non-compact sections with consideration of lateral-torsional buckling, the allowable stress is given by the larger value of the following equations:

$$F_{allow} = \left[2/3 - F_y\left(\frac{L_c}{r_y}\right)^2 /10625000C_b\right]F_y \leq 0.6F_y$$

$$\text{for } \sqrt{708333C_b/F_y} \leq L_c/r_y \leq \sqrt{3541667C_b/F_y} \quad (15)$$

$$F_{allow} = 170000C_b/\left(L_c/r_y\right)^2 \leq 0.6F_y$$

$$\text{for } L_c/r_y \geq \sqrt{3541667C_b/F_y} \quad (16)$$

$$\text{and } F_{allow} = 12000C_b/\left(L_c d_d/A_f\right) \leq 0.6F_y$$

$$\text{for other values of } L_c/r_y$$

Where $C_b$ is the factor corresponding to moment gradient; $A_f$ is the area of compression flange; $d_b$ is the depth of beam. Eq.(17) can be used only for sections with a solid compression flange and approximately rectangular in cross-section and has area not less than the tension flange.

The maximum allowable stress $F_{allow}$ in bending compression as per IS-ASD specification is given by:

$$F_{allow} = 0.66\, f_{cb} F_y \Big/ \left[(f_{cb})^n + (F_y)^n\right]^{1/n} \tag{18}$$

where $f_{cb}$ is the elastic critical stress in bending considering elastic flexural-torsional buckling. and $n = 1.4$.

For bending about weaker axis. the AISC-ASD specification permits a maximum allowable stress in bending compression as:

$$F_{allow} = \begin{cases} 0.75F_y & \text{for compact section} \\ 0.6F_y & \text{for non–compact section} \end{cases} \tag{19}$$

On the other hand, the IS-ASD specifications give a single value of $0.66F_y$ without classifying for compact and non-compact sections.

*AISC-LRFD Specifications:* The AISC-LRFD specifications define the strength limit state for beams as:

$$M_u = \phi\, M_p \tag{20}$$

where $\phi_b$, is the strength reduction factor (0.9) for flexure. This assumes that the section is prevented from lateral-torsional buckling, flange local buckling and web buckling. The moment capacity $M_r$ associated with the limit state corresponding to the first yield strain is defined as

$$M_r = (F_y - F_r)S \tag{21}$$

Corresponding nominal moment capacity $M_u$ is equal to the $\phi_b M_r$ where $F_r$ is an allowance for residual stress and $S$ is the elastic section modulus. The inelastic strength limit state starts at $M_r$, and the upper bound for the inelastic strength limit $M_p$. Thus, the flexural capacity $M_u$ in the inelastic range is a function of the induced strain state (Fig.1) and can be written as:

$$M_u = \phi_b\, C_b\, [M_p - (M_p - M_r)\,\beta] \tag{22}$$

where $M = (L_b - L_p)/(L_r - L_p)$ ; $L_b$, $L_r$ and $L_p$ are the maximum permissible unbraced length of compression flange with respect to the current state of strain, as well as the elastic and plastic state of strain respectively, and can be obtained from stability limit state as discussed in Part I[1]. $C_b$ is the modification factor For non-uniform moment diagram, which can be conservatively taken as 1.0 for all cases When $L_b$ exceeds $L_r$, i.e . within the elastic range, the nominal moment capacity $M_u$ given by:

$$M_u = \frac{\phi_b C_b S X_1 \sqrt{2}}{(L_b/r_y)} \sqrt{1 + \frac{X_1^2 X_2}{2(L_b/r_y)^2}} \leq \phi_b\, M_b \tag{23}$$

where $X_1 = \dfrac{\pi}{S}\dfrac{EGJA}{2}$, $X_2 = \dfrac{4C_w}{I_y}\left(\dfrac{S}{GJ}\right)^2$, $I_y$ is the weaker axis moment of inertia, GJ is the torsional rigidity, and $C_w$ is the warping constant.

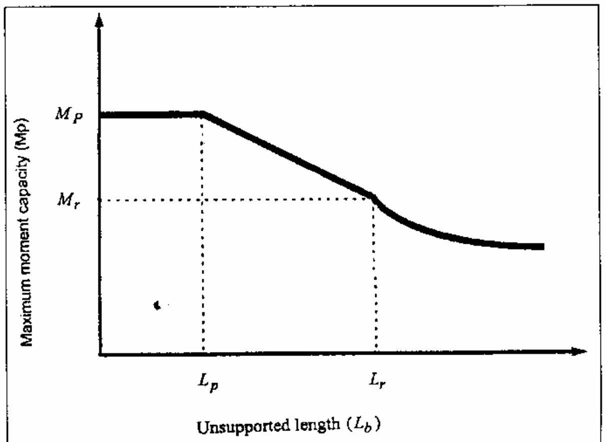


FIG.1 BEAM MOMENT CAPACITY AS FUNCTION OF LATERALLY UNBRACED LENGTH

**Connections Requirements**

In a broad sense, a connection in steel structure refers to the attachment of one structural element to another such that the desired loads can be transferred between them. The types of connections include beam to column attachments bracing connections, gravity connections, and column base plate connections. However, the discussion in this paper is restricted to the moment resisting beam-to-column connections.

The moment resisting connections can be broadly categorised as welded and bolted connections. In the bolted connections, the bolts are designed for axial (tensile or compressive) and shear forces to transfer the moment and shear forces, respectively, across the joint. In welded connections, typically the beam flanges are welded to the column flanges to transfer the bending moment, and beam web is either welded or bolted with column flange to transfer shear forces.

*AISC and IS: Provisions:* As per AISC-ASD and IS-ASD specifications, the design forces obtained from beams are considered in designing of welds or bolts in a moment resisting connection. In Plastic Design approach, the connections are designed for full strength of the beams, namely $M_p$.

*Special Seismic Provisions for Connection Design in American Codes:* The AISC-SPSSB specifications give detailed guidelines for design of moment resisting connections in both ordinary Moment Resistance Frames (OMRF) and Special Moment Resistance Frames (SMRF). The UBC specifications are also identical. The design shear force for connections of both OMRF and SMRF should be obtained from the gravity shear using the load combination 1.2D + 0.5L + 0.2S plus the shear resulting from $1.1R_y$ times plastic moment capacity of the beam. However, this design shear is not required to exceed the shear obtained from load combination as give in Eq.(7).

For transferring flexural forces in both OMRF and SMRF, the beam flanges should be welded to column flanges using full penetration butt weld to develop the full strength of the beam.

**Joint Panel Zone Requirements**

In a moment resisting frame, a joint panel zone is that portion of the column web (or webs) which is effective in developing the flexural stress from the girders, through shear behaviour. Proper design and detailing of the panel zone assumes great importance in ensuring ductile behaviour of a moment-resisting frame. The panel zone must be checked against (a) joint panel shear that comes from the flexural action of the beam(s), (b) panel zone buckling due to direct compressive force from beam flange, and (c) panel zone shear buckling. The column flange strength at panel zone should also be checked for proper transfer of beam flange forces to the panel zone web.

The AISC-LRFD specifications are based on factored forces whereas the SPSSB specifications are based on forces corresponding to plastic capacity of the members. A few of the design issues for panel zone are also discussed in the plastic design method given in IS:800-1984 and in the plastic design handbook SP:6(6)-1972[8].

*Panel Zone Shear Design*: The AISC-LRFD approach for panel zone design is based on two strength criteria. The first one is based on the first-yield strength according to which the joint strength $\phi_v V_n$ is estimated by:

$$\phi_v V_n = \begin{cases} 0.6\phi_v F_y d_c t_z & \text{for } P_u \le 0.4P_y \\ 0.6\phi_v F_y d_c t_z (1.4 - P_u/P_y) & \text{for } P_u > 0.4P_y \end{cases} \tag{24}$$

and the second one is based on the post-yield strength according to which the joint strength $\phi_v V_n$ is given by:

$$\phi_v V_n = \begin{cases} 0.6\phi_v F_y d_c t_z (1 + 3b_{cf} t_{cf}^2 / d_c d_b t_z) & \text{for } P_u \le 0.75P_y \\ 0.6\phi_v F_y d_c t_z (1 + 3b_{cf} t_{cf}^2 / d_c d_b t_z)(1.9 - 1.2P_u/P_y) & \text{for } P_u > 0.75P_y \end{cases} \tag{25}$$

where the $b_{cf}$ and $t_{cf}$ are the column flange width and thickness, respectively; $d_c$ and $d_b$ are column web depth and average beam web depth, respectively; and $t_z$ is the column web thickness including doubler plate, if any, $P_u$ is the design axial load; and $P_y$ is the strength reduction factor, equal to 0.9. For an interior joint panel zone, this strength has to be checked against the joint panel shear $V_u$ given by:

$$V_u = \frac{M_{ul}}{0.95 d_{b1}} + \frac{M_{u2}}{0.95 d_{b2}} - V_c \tag{26}$$

where $M_{u1}$ and $M_{u2}$ are the factored beam moments at the face of joints; $db_1$ and $db_2$ are the beam depths and $V_c$ is the factored column shear.

The AISC-SPSSB specification for joint panel strength uses Eq.(25) with $\phi_v$ = 075 This strength has to be checked against 80% of the shear force obtained from $R_y$ times the plastic moment capacities of the beams framing into the joint.

SP: 6(6)-1972[8] gives the required thickness of joint panel zone of a straight corner joint as:

$$t = \sqrt{3}\, S/d_c^2 \tag{27}$$

where $S$ is the section modulus of the beam. This expression is obtained by equating the joint shear demand corresponding to the beam plastic moment with the panel zone shear strength ($\sqrt{3}\, F_y d_c t_w$).

*Doubler Plate in Joint Panel Zone*: If the joint panel zone thickness is not enough to develop the required shear strength its thickness should be increased by welding an additional plate against the panel zone web (Fig.2). This additional Plate, called as *doubler plate*, should be welded all around the panel web.

*Continuity Plate in Joint Panel Zone*: The AISC-SPSSB specifications for panel zone design rely on both panel zone and beam for energy dissipation under strong seismic shaking. In case of strong earthquake shaking, when beams go into the strain hardening range, panel zone yielding increases, and a large amount of energy dissipation takes place through shear yielding of the panel zone. Thus, unduly large ductility demands on the beams can be avoided if the panel zones are designed to yield. However, experimental results[9] show that, in order to accommodate the large shear deformations that are required to make the panel zone yield, excessive kinking is induced in the column and beam flanges at the joint. To prevent this, the panel zone is reinforced by introducing a continuity plate (Fig.2).

Continuity plates prevent the local buckling of column flange at the beam-column joint. These plates are welded to the flange and web of the column in line with the beam flange, on either side of the column web. AISC specifications estimate the dependable strength, $\phi R_n$ of the column flange as:

$$\phi R_n = \phi\ 6.25 t_{cf}^2\, F_y \tag{28}$$

where the strength reduction factor $\phi$ is 0.9. As per the LRFD specifications, this strength has to be checked against factored force generated in the beam flange. As per SPSSB specifications, the welded joints between the continuity plates and the column web should sustain a design strength not less than the following: (a) sum of design strengths at

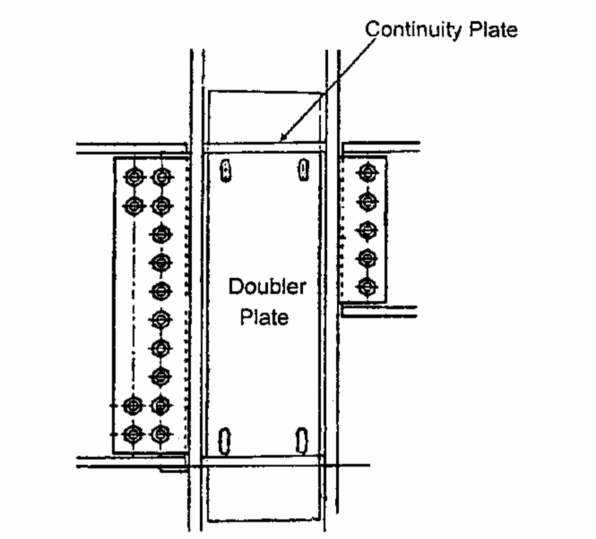


FIG.2 PANEL ZONE REINFORCEMENT USING DOUBLER PLATE AND CONTINUITY PLATE[11]

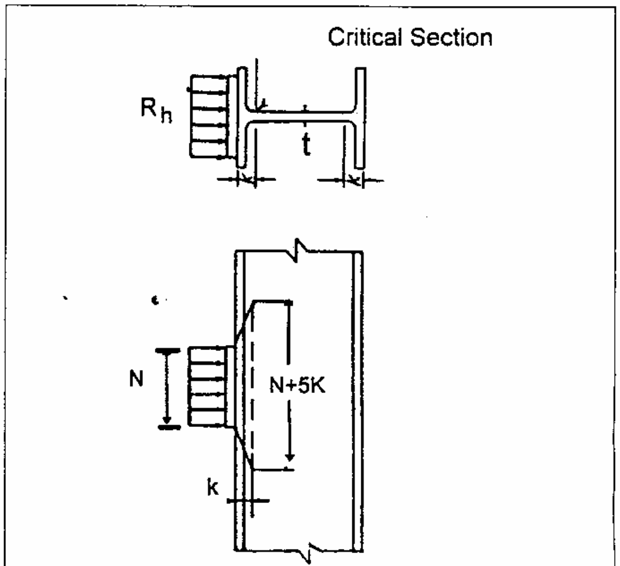


FIG.3 COMPRESSIVE STRENGTH OF COLUMN WEB[12]

the connection of the continuity plate to column flange, (b) design shear strength of the contact area of the plate with the column web, (c) weld design strength that develop the design shear strength of column panel zone, and (d) the actual force transmitted by the stiffener.

*Column Web Yielding at Joint Panel Zone*: Column web may crush or yield due to compressive flange forces offered by beams at beam column joints. Once the beam flange force $P_{bf}$ is safely transmitted to the column flange, this force is transferred to the column web leading to very high compressive stresses in the column web at the toe of the flange. As per AISC-LRFD specifications, the compressive strength $R_n$ of column web is estimated by:

$$R_n = (5k + N)\ F_y t_z \tag{29}$$

where $k$ is the distance from outer face of the flange to the web toe of fillet, and $N$ is the length of bearing (Fig.3). If the dependable strength $\phi R_n$ is less than $P_{bf}$ obtained from factored beam moment, then doubler plates need to be added to the column web in the joint panel zone.

As per IS-PD specifications, the minimum thickness of the panel zone plate required to resist the compressive force from beam flange is gives by:

$$t_z = A_f/(N + 5k) \tag{30}$$

where $A_f$ is the area of the flange delivering the concentrated force. If $t_z$ exceeds the column panel web thickness, the area of stiffening plate $A_{st}$ is given by:

$$A_{st} \geq A_f - t_z\ (N + 5k) \tag{31}$$

*Column Web Shear Buckling*: Both UBC and the AISC-SPSSB specifications identify the minimum thickness $t_z$ of the panel zone plate to maintain the shear capacity of the panel zone under cyclic loading as:

$$t_z \geq (d_c + d_b)/90 \tag{32}$$

where $d_c$ and $d_b$ are the depth of column and beam within the joint panel zone, respectively.

*Welding Detail for Doubler Plate*: The centre of the doubler plate (Fig.4) is subjected to pure shear, whereas the region around the centre has a combination of shear and tension or compression developed from the flange forces. Vertical welds should be designed such that, they can develop full strength of the doubler plate. The horizontal welds should be designed to carry the shear force given by the column shear $V_c = (M_{p1} + M_{p2})/h$ times the ratio of doubler plate thickness to column web panel thickness; here, $h$ is the average storey height above and below the joint.

*Relative Strengths of Beams Columns*: As per SPSSB specifications following LRFD approach, the strengths of the beam, column and panel zone shall be governed by the following strength hierarchy to ensure *strong-column weak-beam* behaviour

$$\Sigma Z_c\ (F_y - P_{uc}/A_g)\ /\Sigma(1.1R_y M_p + M_y) \geq 1.0 \tag{33}$$

where $A_g$ is the gross area of a column; $F_y$ is yield stress; $P_{uc}$ is required axial strength of column; and $M_y$ is the additional moment due to shear amplification from the location of plastic hinge to the column centreline.

However, the code specifies that these requirements need not be applied for columns with $P_{uc} < 0.3F_y A_g$, (a) columns in any storey that have a ratio of design shear strength to design shear force 50% greater than that in the storey above, (b) columns which are not designed to trans-

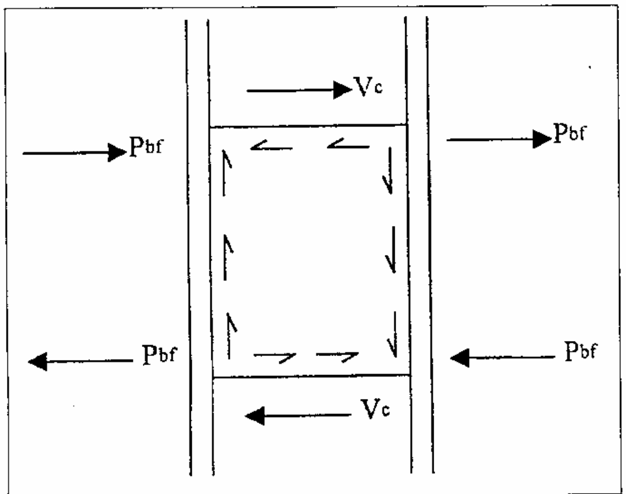


FIG.4 FORCES ACTING ON THE COLUMN WEB AND DOUBLER PLATE

fer seismic shear, but only to resist the overturning axial force, and (c) columns of a one-storey building or in the top storey of a multi-storey building.

## DRIFT CRITERIA

In most of the moment resisting frames (MRF) of moderate to large height, the final design is generally governed by drift limitation. This is evident from the fact that in the design of such MRF the final sizes of most members need to be revised to limit the lateral deflection within code specified values. Thus it is necessary to have a proper method of drift estimation. For this reason UBC 97 guidelines for estimating drift in MRF suggest that drift calculations should include both bending and shear contributions of the girders and columns. In doing so, rigid joints, axial deformation of columns, and rotation and distortion of joint panel zone are to be considered. UBC 97 also permits a simple process of drift calculation using centre line method. if (a) the drift obtained by this method is within 15% of that obtained considering all the above contributions, or (b) the nominal panel zone strength given by AISC-SPSSB specification is equal to or greater than $0.8\Sigma\ M_p$ of girders framing into the column flanges at that joint. The allowable storey drift under design seismic force as per UBC 97 is given as:

$$\Delta_{allow} = \begin{cases} 0.04\ h/R \leq 0.005h & \text{for } T < 0.7sec \\ 0.03\ h/R \leq 0.004h & \text{for } T > 0.7sec \end{cases}$$

where $h$ is the storey height and $R$ is the response reduction factor. Thus, allowable drift depends upon the flexibility of the structure and response reduction factor.

The maximum permissible inter-storey drift due to design seismic forces is specified in IS:1893-1984[10] as 0.004 times the storey height. This provision does not distinguish between structures with different detailing or with different natural periods. Further the code does not state any drift estimation procedure. Hence, the estimated drift depends on how the designer models the frame. In case of centre line modelling, the flexural deformation of joint panel zone is considered in terms of flexural deformation of beam element. But, in practice the joint panel zone is rigid against flexural deformation. On the other hand, it undergoes significant amount of shear deformation, which is usually neglected. In case of rigid joint panel zone modelling, the estimated deformation is always less than the actual one. Thus, the current Indian code is inadequate in the drift provisions and more attention should be paid on drift estimation of MRF.

## SUMMARY AND CONCLUSIONS

The following aspects are not addressed by the current Indian codes: the strength hierarchy of beam and column; design and detailing of joint panel zone; drift estimation procedure for MRF. The salient features of the American and Indian codes pertaining to strength and drift criteria for seismic design of steel structures are presented here:

1. In the Indian codes, design of the joint panel zone should be considered more formally. The IS-PD has some guidelines for designing joint panel zone and for the stiffening plates. However, absence of proper commentary regarding joint behaviour under seismic loading, fails to motivate the designers to adopt it. A properly designed joint panel zone can absorb a lot of earthquake energy. But, at the same time, the joint panel zone deformations have to be evaluated and excessive deformations should be controlled by using doubler plates. Both the continuity and doubler plates at a joint panel zone, should be designed for the extreme forces that may be transferred through the joint.

2. The strong-column weak-beam concept of frame design as adopted by the American design practice is very important in ensuring ductile behaviour of the frame. The Indian codes have no such specifications. Further, when the final design is governed by the specified drift limits, arbitrarily increasing the beam or column sections may lead to a structure with a poor failure mechanism.

3. Proper drift estimation is one of the major aspects in the final design of a moment resisting frame. In American codes, clear guidelines are available regarding the importance of including joint deformation in estimating the drift of a moment resisting frame. The drift estimation procedure must be clearly outlined in the Indian code. The sizing of members should be such that a generally uniform storey drift is obtained. This assures a uniform energy absorption capacity along the height of the structure and reduces the excessive ductility demand on a few critical elements of the structure.

4. The estimation of drift in an MRF shall be based on the analysis of the structure considering (a) flexural and shear deformations of the frame members, and (b) finite size and stiffness of the joint panel zones.

## ACKNOWLEDGEMENTS

This study forms part of a project on earthquake resistant building construction and education sponsored by the Ministry of Human Resource Development at the Indian Institute of Technology, Kanpur. The authors are grateful for financial support and encouragement''.

## REFERENCES

1. Paul, S., Murty, C.V.R., and Jain, S.K., "State of-the-Art Review of Seismic Design of Steel Moment Resisting Frames, Part I: General Considerations and Stability Provisions'', *J. Struct. Engg. SERC*, Vol.27, No. I April 2000, pp.23-32.

2. AISC-ASD and AISC-PD, ''*Allowable Stress Design and Plastic Design Specifications for Structural Steel Buildings*'', American Institute of Steel Construction, Chicago, USA, 1989.

3. AISC-LRFD, ''*Metric Load and Resistance Factor Design Specifications for Structural Steel Buildings*'', American Institute of Steel Construction, Chicago, USA, 1997.

4. ''*NEHRP Recommended Provisions for Seismic Regulations for New Buildings*'', Federal Emergency Management Agency, Washington, D.C., 1997.

5. ''*Uniform Building Code*'', International Conference of Building Officials, Whittier, CA, 1997.

6. AISC-SPSSB, ''*Seismic Provisions for Structural Steel Buildings*'', American Institute of Steel Construction, Chicago, 1997.

7. IS:800-1984: ''*Indian Standard Code of Practice for General Construction in Steel*'', Bureau of Indian Standards, New Delhi, 1985.

8. SP:6(6)-1972: ''*Application of Plastic Theory in Design of Steel Structures*'', Indian Standards Institution, New Delhi, 1973.

9. Krawinkler, H., Bertero, V.V., and Popov, E.P., ''Shear Behaviour of Steel Frame Joints'', *J. Struct. Div. ASCE*, Vol. 101, No.ST11, pp.2317-2334, 1975.

10. IS:1893-1984: ''*Indian Standard Criteria for Earthquake Resistant Design of Structures*'', Bureau of Indian Standards, New Delhi, 1984.

11. Englekirk, R., ''*Steel Structures Controlling Behaviour Through Design*'', John Wiley and Sons, New York, 1993.

12. Salmon C.G., and Johnson, J.E., ''*Steel Structures Design and Behaviour*'', Third Edition, Harper Collins Publishers Inc., 1990.