12.1 General

Steel frames shall be so designed and detailed as to give them adequate strength, stability and ductility to resist severe earthquakes in all zones classified in IS:1893 (Part 1) without collapse. Frames which form a part of the gravity load resisting system but are not intended to resist the lateral earthquake loads, need not satisfy the requirements of this section, provided they can accommodate the resulting deformation without premature failure.

Steel frames in a structural system may be assigned as follows:

- Frames which resist severe earthquake loads
- Frames which donot resist earthquake loads

Steel frames which are to be designed and detailed to resist severe earthquake loads shall be designed and detailed to have a) adequate strength, b) stability, c) ductility in all zones which are classified in IS: 1893 so that it doesnot collapse. Those frames of a framing system which are intended to resist the lateral earthquake loads need to satisfy the requirements stated in this chapter. Rest frames which are a part of the structural system resisting the gravity load resisting system need not satisfy the requirements of this section. They should only resist the resulting deformation without failing prematurely.

12.2 Load and Load Combinations

12.2.1 Earthquake loads shall be calculated as per IS: 1893 (Part 1), except that the reduction factors recommended in 12.3 may be used.

IS: 1893 Part 1 (Criteria for Earthquake Resistant Design of Structures) suggests methods of calculating the earthquake loads. The Response Reduction factors given in Table 7 of IS 1893 are to be followed in all situations except in cases of

a) Braced Frame system (for OCBF, SCBF, EBF) and
b) Moment Frame system (for OMF and SMF).

For these cases the Response Reduction Factor should be considered as per Table 23 of the Code.

However Table 7 of IS: 1893 deals with Concentric, Eccentric and Moment resisting frames in Steel. Table 23 deals in much detail on the same types of frames.

Conclusion: For Steel frames Table 23 covers most of the types of frames.

12.2.2 In the limit state design of frames resisting earthquake loads, the load combinations shall be conform to Table 4.

<table>
<thead>
<tr>
<th>Combination</th>
<th>Limit State of Strength</th>
<th>Limit State of Serviceability</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL+LL+CL+EL</td>
<td>DL Leading: 1.2</td>
<td>DL Leading: 1.0</td>
</tr>
<tr>
<td></td>
<td>LL Accompanying: 0.53</td>
<td>LL Accompanying: 0.8</td>
</tr>
<tr>
<td></td>
<td>EL: 1.2</td>
<td>EL: 0.8</td>
</tr>
</tbody>
</table>

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### 12.2.3 In addition, the following load combination shall be considered as required in 12.5.1.1, 12.7.3.1, 12.11.2.2 and 12.11.3.4.

a) 1.2DL + 0.5 LL ± 2.5 EL

b) 0.9DL ± 2.5 EL

<table>
<thead>
<tr>
<th>Combination</th>
<th>Limit State of Strength</th>
<th>DL</th>
<th>LL</th>
<th>EL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Leading</td>
<td>Accompanying</td>
<td></td>
</tr>
<tr>
<td>DL+LL+CL+ EL</td>
<td>1.2</td>
<td>0.5</td>
<td>—</td>
<td>2.5</td>
</tr>
<tr>
<td>DL+LL+CL- EL</td>
<td>1.2</td>
<td>0.5</td>
<td>—</td>
<td>(-) 2.5</td>
</tr>
<tr>
<td>DL+ EL</td>
<td>0.9</td>
<td>—</td>
<td>—</td>
<td>2.5</td>
</tr>
<tr>
<td>DL- EL</td>
<td>0.9</td>
<td>—</td>
<td>—</td>
<td>(-) 2.5</td>
</tr>
</tbody>
</table>

Please refer to the commentary on the corresponding clauses 12.5.1.1, 12.5.1.2, 12.11.2.2, 12.11.3.4

### 12.3 Response Reduction Factor

For structures designed and detailed as per the provision of this section, the response reduction factors specified in Table 23 may be used in conjunction with the provision in IS: 1893 for calculating the design earthquake forces.
TABLE 23 RESPONSE REDUCTION FACTOR $R$ FOR BUILDING SYSTEM
(Section 23)

<table>
<thead>
<tr>
<th>SI.No</th>
<th>Lateral Load Resisting System</th>
<th>$R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>i)</td>
<td><em>Braced frame systems:</em></td>
<td></td>
</tr>
<tr>
<td></td>
<td>a) Ordinary Concentrically Braced Frames (OCBF)</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>b) Special Concentrically Braced Frame (SCBF)</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>c) Eccentrically Braced Frame (EBF)</td>
<td>5</td>
</tr>
<tr>
<td>ii)</td>
<td><em>Moment Frame System:</em></td>
<td></td>
</tr>
<tr>
<td></td>
<td>a) Ordinary Moment Frame (OMF)</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>b) Special Moment Frame (SMF)</td>
<td>5</td>
</tr>
</tbody>
</table>

As specified in Clause 12.2.1 the Response Reduction factors given in Table 7 of IS 1893 are to be followed in all situations except in cases of:
- a) Braced Frame system (for OCBF, SCBF, EBF) and
- b) Moment Frame system (for OMF and SMF).

For these cases the Response Reduction Factor should be considered as per Table 23 of the Code.

However Table 7 of IS: 1893 deals with Concentric, Eccentric and Moment resisting frames in Steel. Table 23 deals in much detail on the same types of frames.

Conclusion: Table 23 covers most of the types of frames. So that table becomes guiding.

12.4 Connections, Joints and Fasteners

12.4.1 All bolts used in frames designed to resist earthquake loads shall be fully tensioned High Strength Friction Grip (HSFG) bolts or turned and fitted bolts.

| HSFG bolts conforming to IS: 4000 or Turned and Fitted Bolts conforming to IS 1364 to be used for connection. The bolts are to be fully tensioned. |

12.4.2 All welds used in frames designed to resist earthquake loads shall be complete penetration butt welds, except in column splice, which shall confirm to 12.5.2.

| All welds should be full penetration butt weld except for column splice. No other type of weld (ie fillet, spot, intermittent etc.) is allowed |

12.4.3 Bolted joints shall be designed not to share load in combination with welds on the same faying surface.

| A joint which is connected by bolts and welds do not share the loads between bolts and welds simultaneously. Forces are transferred either totally by bolts or totally by welds. Thus bolts are to be separately designed to transfer the total forces. Same for the welds |
12.4 Columns

12.5.1 Column Strength

When \( P_r/P_d \) is greater than 0.4, the requirements in 12.5.1.1 and 12.5.1.2 shall be met where

\[ P_r = \text{required compressive strength of the member} \]
\[ P_d = \text{design stress in axial compression as obtained from 7.1.2} \]

\[ P_r = \text{required Compressive strength of the member} \]

Or in other words \( P_r = \) the actual factored Comp. load acting on the column from the load combinations specified in table 4 of IS : 800 – 2007

\[ P_d = \text{Design strength in axial compression} = A_e f_{cd} \text{ (Clause 7.1.2)} \]

12.5.1.1 The required axial compressive and axial tensile strength in the absence of applied moment, shall be determined from load combination in 12.2.3

\[
\begin{align*}
\text{i) } DL + LL + CL + EL &= 1.2 DL + 1.2 LL + 0.53 CL + 1.2 EL \\
\text{ii) } DL + LL + CL - EL &= 1.2 DL + 1.2 LL + 0.53 CL - 1.2 EL \\
\text{iii) } DL + EL &= 1.5 DL + 1.5 EL \\
\text{iv) } DL - EL &= 1.5 DL - 1.5 EL
\end{align*}
\]

Applied moment is zero only for frames OCBF and SCBF

12.5.1.2 The required strength determined in 12.5.1.1 need not exceed either of the maximum load transferred to the column considering 1.2 times the nominal strength of the connecting beam or brace element, or the resistance of the foundation to uplift.

\[
\text{The actual factored compressive strength / actual factored tensile strength should be less than either of the max :}
\]

\[
\begin{align*}
\text{i) } Pr &\leq 1.2 \times \text{nominal strength of the connectivity beam i.e. } 1.2 \times Z_p f_y \\
\text{ii) } Pr &\leq 1.2 \times \text{nominal strength of the brace element } P_d = 1.2 \times A_e f_{y} / \gamma_m \text{ or } 1.2 \times A_e f_{cd} \\
\text{iii) } Pr &\leq \text{Resistance of the foundation to uplift}
\end{align*}
\]

If a) \( P_r / P_d > 0.4 \) and has external BM or
b) \( P_r / P_d < 0.4 \) and has external BM or
c) \( P_r / P_d < 0.4 \) and has no external moment

then factored axial compressive load \( (P_r) \) will be as derived from load combinations given in Table 4 of IS 800 – 2007.

NB: DL indicates Dead Load, LL indicates Live Load, EL indicates Earthquake Load

12.5.2 Column splice

12.5.2.1 A Partial-joint penetration groove weld may be provided in column splice, such that the design strength of the joints shall be at least equal to 200 percent of the required strength.

Partial penetration butt weld is allowed in column splices. \( P_d \geq 2 \times P_r \) of the joint

12.5.2.2 The minimum required strength for each flange splice shall be 1.2 times \( f_y A_f \) as shown Fig 20 where \( A_f \) is the area of each flange in the smaller connected column

\[
P_{\text{min}} = 1.2 f_y A_f
\]

Fig 20 Partial penetration groove weld in column splice

The size of the Flange splice plate maybe determined by using the equation \( P_{\text{min}} = 1.2 \times f_y \times A_f \), where \( P_{\text{min}} \) is the minimum strength of the flange splice plate.

12.6 Storey Drift

The storey drift limits shall conform to IS: 1893. The deformation compatibility of members not designed to resist seismic lateral load shall also conform to IS: 1893 (Part 1).

Clause 7.11 of IS 1893 deals with this issue
Max. drift = 0.004 x story height.
Max drift to be calculated against min. specified design lateral Eq force with a Partial factor of safety 1.0
12.7 Ordinary Concentrically Braced Frames (OCBF)

Ordinary concentrically braced frames (OCBF) should be shown to withstand inelastic deformation corresponding at a joint rotation of 0.02 radians without degradation in strength and stiffness below the full yield value. Ordinary concentrically braced frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformation.

*Frame should withstand inelastic deformation corresponding at a joint rotation of 0.02 radians without degradation in strength and stiffness below the full yield value.*
12.7.1.1 Ordinary concentrically braced frames shall not be used in seismic zones IV and V and for buildings with importance factor greater than unity \((I > 1.0)\) in seismic zone III.

**OCBF shall not be used in seismic zones IV and V and for buildings with \(I > 1.0\) in seismic zone III.**

12.7.1.2 The provision in this section apply for diagonal and X- bracing only. Specialist literature may be consulted for V and inverted-V type bracing. K- bracing shall not be permitted in systems to resist earthquake.

**Diagonal and X- bracing only permitted. K- bracing not permitted**

*Specialist literature may be consulted for V and inverted-V type bracing.*

12.7.2 Bracing Members

12.7.2.1 The slenderness of bracing members shall not exceed 120.

\[ \frac{L}{\tau_{\text{min}}} \leq 120 \]

12.7.2.2 The required compressive strength of bracing member shall not exceed 0.8 times \(P_d\) where \(P_d\) is the design strength in axial compression (see 7.1.2)

\[ P_r \leq 0.8 \times P_d \quad P_d = A_e \times f_{cd} \]

12.7.2.3 Along any line of bracing, braces shall be provided such that for lateral loading in either direction, the tension braces will have to resist between 30 to 70 percent of the total lateral load.

12.7.2.4 Bracing cross-section can be plastic, compact or semi-compact, but not slender as defined in 3.7.2.

**Bracing cross-section can be plastic, compact or semi-compact, but not slender**

12.7.2.5 For all built-up braces, the spacing of tack fasteners shall be such that the unfavorable slenderness ratio of individual element, between such fasteners, shall not exceed 0.4 times the governing slenderness ratio of the brace itself. Bolted connections shall be avoided within the middle one-fourth of the clear brace length (0.25 times the length in the middle).

12.7.2.6 The bracing members shall be designed so that gross area yielding (see 6.2) and not the net area rupture (see 6.3) would govern the design tensile strength.

12.7.3 Bracing Connections

12.7.3.1 End connections in bracings shall be designed to withstand the minimum of the following

- A tensile force in the bracing equal to \(1.1 f_y A_g\).
b) Force in the brace due to load combinations in 12.2.3: and
c) Maximum force that can be transferred to the brace by the system.

<table>
<thead>
<tr>
<th>Shall be designed to withstand the minimum of the following</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) ( T_g = 1.1 \times f_y A_g ).</td>
</tr>
<tr>
<td>b) The force in the brace due to load combinations in 12.2.3</td>
</tr>
<tr>
<td>c) The maximum force that can be transferred to the brace by the system.</td>
</tr>
</tbody>
</table>

12.7.3.2 The connection should be checked for tension, rupture and block shear under the load determined in 12.7.3.1.

12.7.3.3 The connection shall be designed to withstand a moment of 1.2 times the full plastic moment of the braced section about the buckling axis.

12.7.3.4 Gusset plates shall be checked for buckling out of their plane.

**Special Concentrically Braced Frames (SCBF)**

12.8 Special Concentrically Braced Frames (SCBF)

12.8.1 Special concentrically braced frames (SCBF) should be shown to withstand inelastic deformation corresponding to a joint rotation of at least 0.04 radians without degradation in strength and stiffness below the full yield value. Special concentrically braced frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformations.

SCBF to withstand inelastic deformation corresponding to a joint rotation of **0.04 radians** without degradation in strength and stiffness below the full yield value

12.8.1.1 Special concentrically braced frames (SCBF) may be used in any seismic zone (see IS: 1893 (Part 1)) and for any buildings (Importance-factor value).

SCBF may be used in any seismic zone for any buildings (I-values).
12.8.1.2 The provision in this section apply for diagonal and X-bracing only. Specialist literature may be consulted for V and inverted V-type bracing. K bracing shall not be permitted in system to resist earthquake.

**Diagonal and X-bracing only permitted, K bracing is not permitted.**  
Specialist literature may be consulted for V and inverted - V type bracing.

12.8.2 Bracing Members

12.8.2.1 Bracing members shall be made of E250B steel of IS 2062 or steel meeting the requirement of Charpy E > 27J.

12.8.2.2 The slenderness of bracing members shall not exceed 160 (only hangers)

\[ \frac{L}{r_{\text{min}}} \leq 160 \]

12.8.2.3 The required compressive strength of bracing member shall not exceed the design strength in axial compression \( P_d \) (see 7.1.2).

\[ P_r \leq P_d \quad \text{and} \quad P_d = A_e \times f_{cd} \]

12.8.2.4 Along any line of bracing, braces shall be provided such that for lateral loading in either direction, the tension braces will resist between 30 to 70 percent of the load.

12.8.2.5 Braced cross-section shall be plastic as defined in 3.7.2.

Section shall be **plastic. Compact, Semi-compact, slender sections are not permitted**

12.8.2.6 In built-up braces, the spacing of tack connections shall be such that the slenderness ratio of individual element between such connections shall not exceed 0.4 times the governing slenderness ratio of the brace itself. Bolted connections shall be avoided within the middle one-fourth of the clear brace length (0.25 times the length, in the middle)

12.8.2.7 The bracing members shall be designed so that gross area yielding (see 6.2) and not the net area rupture (see 6.3) would govern the design tensile strength.

12.8.3 Bracing Connections

12.8.3.1 Bracing end connections shall be designed to withstand the minimum of the following,

a) A tensile force in the bracing equal to \(1.1 f_y A_g\): and

b) Maximum force that can be transferred to the brace by the system.

**Shall be designed to withstand the minimum of the following**

a) \( T_g = 1.1 f_y A_g \).

b) **The maximum force that can be transferred to the brace by the system.**
12.8.3.2 The connection should be checked for tension rupture and block shear under the load determined in 12.8.3.1.

12.8.3.3 The connection shall be designed to withstand a moment of 1.2 times the full plastic moment of the braced section about the critical buckling axis.  

\[ M_d = 1.2 \times M_{pl} \]

12.8.3.4 Gusset plates shall be checked for buckling out of their plane.

12.8.4 Column

12.8.4.1 The column sections used in Special concentrically braced frames (SCBF) shall be plastic as defined in 3.7.2.

The column sections shall be plastic

12.8.4.2 Splices shall be located within the middle one third of the column clear height. Splices shall be designed for the forces that can be transferred to it. In addition, splices in columns shall be designed to develop at least the nominal shear strength of the smaller connected member and 50 percent of the nominal flexural strength of the smaller connected section.

Eccentrically Braces Frames (EBF)

Eccentric Bracing systems in Steel Frames

12.9 Eccentrically Braces Frames (EBF)

12.9.1 Eccentrically braces frames (EBF) shall be designed in accordance with Specialist literature.

Ordinary Moment Frames (OMF)
12.10 Ordinary Moment Frames (OMF)

12.10.1 Ordinary moment frames (OMF) should be shown to withstand inelastic deformation corresponding to a joint rotation of 0.02 radians without degradation in strength and stiffness below the full yield value ($M_p$). Ordinary moment frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformation.

OMF should be shown to withstand inelastic deformation corresponding to a joint rotation of 0.02 radians without degradation in strength and stiffness below the full yield value ($M_p$).

12.10.1.1 Ordinary moment frames (OMF) shall not be used in seismic zones IV and V and for buildings with importance factor greater than unity ($I > 1.0$) in seismic zone III.

12.10.2 Beam -to-Column Joints and Connections

Connections are permitted to be Rigid or Semi-rigid moment connections and should satisfy the criteria in 12.10.2.1 to 12.10.2.5.

Rigid or Semi-rigid moment connections

12.10.2.1 Rigid moment connections should be designed to withstand a moment of at least 1.2 times either the full plastic moment of the connected beam or the maximum moment that can be delivered by the beam to the joint due to the induced weakness at the end of the beam, whichever is less.
12.10.2.2 Semi-rigid connections should be designed to withstand either a moment of at least 0.5 times the full plastic moment of the connected beam or the maximum moment that can be delivered by the system, whichever is less. The design moment shall be achieved within a rotation of 0.01 radians. The information given in Annex F may be used for checking.

12.10.2.3 The stiffness and strength of semi-rigid connections shall be accounted for in the design and the overall stability of the frame shall be ensured.

12.10.2.4 The Rigid and Semi-rigid connections should be designed to withstand a shear resulting from the load combination $1.2DL + 0.5LL$ plus the shear corresponding to the design moment defined in 12.10.2.1 and 12.10.2.2, respectively.

12.10.2.5 In Rigid fully welded connections, continuity plates (tension stiffener, see 8.7) of thickness equal to or greater than the thickness of the beam flange shall be provided and welded to the column flanges and web.

### Special Moment Frames (SMF)

Special Moment Resisting Framing systems in Steel Frames

12.11 Special Moment Frames (SMF)

12.11.1 Special moment frames (SMF) shall be made of E 250 B steel of IS: 2062 or steel meeting the charpy E>27J and should be shown to withstand inelastic deformation.
corresponding to a joint rotation of 0.04 radians without degradation in strength and stiffness below the full yield value ($M_p$). Special moment frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformation.

Steel of Grade E 250 B of IS 2062 or steel meeting with Charpy E >27J

12.11.1 Special moment frames (SMF) may be used in any seismic zone (see IS: 1893 (Part 1)) for any buildings (Importance – factor values).

12.11.2 Beam-to-Column Joints and Connections

12.11.2.1 All beam-to-column connections shall be rigid see Annex F and designed to withstand a moment of at least 1.2 times the full plastic moment of the connected beam. When a reduced beam section is used, its minimum flexural strength shall be at least equal to 0.8 times the full plastic moment of the unreduced section.

12.11.2.2 The connection shall be designed to withstand a shear resulting from the load combination \(1.2 DL + 0.5 LL\) plus the shear resulting from the application of \(1.2 M_p\) in the same direction, at each end of the beam (causing double curvature bending). The shear strength need not exceed the required value corresponding to the load combination in 12.2.3.

Rigid connections should be designed to withstand

1. \(V_d\) = shear developed by the load combination of \(1.2 \, DL + 0.5 \, LL + \text{shear developed by} \, 1.2 \, M_p\) of beam.

12.11.2.3 In column strong axis connections (beam and column web in the same plane), the panel zone shall be checked for shear buckling in accordance with 8.4.2 at the design shear defined in 12.11.2.2. Column web doubler plates or diagonal stiffeners may be used to strengthen the web against shear buckling.

12.11.2.4 The individual thickness of the column webs and doubler plates, shall satisfy the following

\[
t \geq (d_p + b_p) / 90
\]

where

- \(t\) = thickness of column web or doubler plate
- \(d_p\) = panel -zone depth between continuity plate, and
- \(b_p\) = panel-zone width between column flanges
Interpretation. Chapter 12. IS:800-2007

**FIG 21 CONTINUITY PLATES**

12.11.2.5 Continuity plates (tension stiffeners) (see 8.7) shall be provided in all strong axis welded connections except in end plate connection.

12.11.3 Beam and Column Limitation

12.11.3.1 Beam and column sections shall be either plastic or compact as defined in 3.7.2. At potential plastic hinge locations, they shall be necessarily plastic.

12.11.3.2 The section selected for beams and columns shall satisfy the following relation:

\[
\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.2
\]

where

\[\sum M_{pc}\] = sum of the moment capacity in the column above and below the beam centerline; and

\[\sum M_{pb}\] = sum of the moment capacity in the beams at the intersection of the beam and column centerlines.

In tall buildings, higher mode effects shall be accounted for in accordance with specialist literature.

12.11.3.3 Lateral support to the column at both top and bottom beam flange levels shall be provided so as to resist at least 2 percent of the beam flange strength, except for the case described in 12.11.3.4.

12.11.3.4 A Plane frame designed as non sway in the direction perpendicular to its plane, shall be checked for buckling, under the load combination specified in 12.2.3.

12.12 Column Bases

12.12.1 Fixed column bases and their anchor bolts should be designed to withstand a moment of 1.2 times the full plastic moment of the column section. The anchor bolts shall be designed to withstand the combined action of shear and tension as well as prying action if any.

<table>
<thead>
<tr>
<th>Design of BASE PLATE and HD Bolt for Fixed joint under bending:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Design moment = 1.2 x Mp of the column = 1.2 x (Zp x fy) of the column</td>
</tr>
<tr>
<td>Load combination for anchor bolt</td>
</tr>
<tr>
<td>a) Shear + Tension b) prying force, if any</td>
</tr>
</tbody>
</table>

12.12.2 Both fixed and hinges column bases shall be designed to withstand the full shear under any load case or 1.2 times the shear capacity of the column section whichever is higher.

<table>
<thead>
<tr>
<th>Design of BASE PLATE for Fixed/ Hinged joint under shear:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Design shear = Actual horizontal shear at the underside of base plate</td>
</tr>
<tr>
<td>2. Design shear = 1.2 x (Av x fy/√3) of the column = 1.2 x shear capacity of col.</td>
</tr>
<tr>
<td>whichever is higher. Observation: In most situations Sl. No 2 will be higher.</td>
</tr>
</tbody>
</table>
Prepared by Mr M M Ghosh-AGM (C&S), INSDAG

References:

- Teaching Resource Material-INSDAG

Acknowledgement:

- Dr S R Satishkumar-Professor, IIT Madras