भारतीय मानक

Indian Standard

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इस्पात भवनों का भूकंप प्रतिरोधी डिजाइन और विवरण — रीति संहिता

Earthquake Resistant Design and Detailing of Steel Buildings — Code of Practice

ICS 91.120.25

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Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

Many regions of India are prone to strong earthquake shaking, and hence earthquake resistant design of buildings and structures is essential. Buildings designed as per this standard are expected to sustain damage during strong earthquake ground shaking. The provisions of this standard are intended for earthquake resistant design of only normal buildings (without energy dissipation devices or systems in-built). To control loss of life and property, base isolation or other advanced techniques may be adopted. A new Indian Standard IS 1893 (Part 6) : 2022 'Criteria for earthquake resistant design of structures: Part 6 Base isolated buildings' was published for design of such buildings.

Provisions for ductile design and detailing of steel structures were first published in the country as Section 12 of IS 800 : 2007 'General construction in steel — Code of practice (*third revision*)'. But, a need was felt for having a separate and detailed standard dealing only with earthquake resistant design requirements of steel buildings in line with IS 13920 : 2016 'Ductile design and detailing of reinforced concrete structures subjected to seismic forces — Code of practice (*first revision*)'. Towards this cause, the National Disaster Management Authority offered financial assistance for research on the subject, which the Committee and BIS acknowledges.

This standard contains provisions specific to earthquake resistant design of steel buildings using moment or braced frames as lateral load resisting systems. In particular, provisions related to mechanical properties of steel and welding electrodes, design of beams, columns and joints in special moment resisting frames, and design of braces, shear links, and joints in braced frames are covered. Thus, provisions of this standard shall govern when in conflict with those in Section 12 of IS 800 : 2007.

In the development of this standard, effort has been made to coordinate with standards and practices prevailing in different countries in addition to relating them to the practices in the field in this country. Assistance has particularly been derived from the following publications:

> ANSI/AISC 360-16, 'Specification for Structural Steel Buildings', American Institute of Steel Construction, Chicago, IL, USA, 2016
>
> ANSI/AISC 341-16, 'Seismic Provisions for Structural Steel Buildings', American Institute of Steel Construction, Chicago, IL, USA, 2016

The compostion of the Committee responsible for the formulation of the standard is given in Annex B.

For the purpose of deciding whether a particular requirement of this standard is complied with the final, observed or calculated value, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

EARTHQUAKE RESISTANT DESIGN AND DETAILING OF STEEL BUILDINGS — CODE OF PRACTICE

1 SCOPE

1.1 This standard covers the requirements for designing and detailing of structural components and members of steel buildings designed to resist lateral effects of earthquake shaking, so as to provide them with adequate stiffness, strength and ductility to resist severe earthquake shaking without collapse. The general concepts adopted in this standard for buildings are applicable also for other types of structures; in particular, provisions of this standard may be taken as a guide for design of components of industrial structures and steel bridges.

1.2 Provisions of this standard shall be adopted in the design of the following types of steel buildings located in seismic zones III, IV or V of IS 1893 (Part 1):

a) Residential, educational and institutional buildings;
b) Office and business buildings; and
c) Community, utility and lifeline buildings required for disaster management activities.

The use of this standard is optional for design of steel buildings located in seismic zone II.

1.3 The provisions of this standard apply for design and detailing of steel buildings having the following structural systems:

a) Special moment resisting frame (SMRF);
b) Special concentrically braced frame (SCBF); and
c) Eccentrically braced frame (EBF).

In seismic zone V, all steel buildings shall be made of EBF systems; SCBFs shall not be used. In seismic zones IV and V, SMRFs may be used in buildings of height less than 15 m.

1.4 All steel frames in a building need not be designed to resist earthquake induced lateral loads. The designer can identify judiciously select frames to resist together the vertical loads and at least 80 percent of the effects of design earthquake lateral load and designate them as the lateral load resisting system. The other frames shall be designed to resist the effects imposed on them due to vertical loads and the deformation compatibility induced effects of earthquake shaking. Further, moments shall not be transferred to the designated lateral load resisting systems from frames spanning in perpendicular directions. Consequences of failure of structural members that are not part of the lateral load resisting system shall also be considered in design.

2 REFERENCES

The standards listed in Annex A contain provisions which, through reference in this standard, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreement based on this standard are encouraged to investigate the possibility of applying the most recent editions of these standards.

3 TERMINOLOGY

For the purpose of this standard, the terms given in IS 1893 (Part 1) and the following definitions shall apply:

3.1 Beams — Members (generally horizontal) resisting loads through flexural and shearing actions.

3.2 Braces — Members (generally inclined) resisting loads through axial actions.

3.3 Capacity Design — The design steps (beyond those in the basic design of non-yielding structural elements, members and connections), which consider the effects of inelasticity incurred in yielding members resulting in overstrength-based demands on the non-yielding structural elements.

3.4 Capacity Protected Elements — Components or members designed to remain elastic using capacity design principles when an adjacent component or member undergoes inelastic straining during design earthquake shaking.

3.5 Collector — A member that serves to transfer loads between diaphragms and the members of the vertical force-resisting elements of the lateral load resisting system.

3.6 Columns — Members (generally vertical) resisting loads through axial, flexural and shearing actions.

3.7 Column Bases — Assembly of columns, plates, connector weld or bolts, and anchor rods at the base of columns designed to transmit forces from the steel superstructure to reinforced concrete pedestal or foundation.

3.8 Concentrically Braced Frame (CBF) — A lateral load resisting system composed of interconnected beams and columns with inclined members as braces, which function as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resists gravity and lateral force effects primarily by axial actions.

3.8.1 *Special Concentrically Braced Frame* (*SCBF*) — A CBF specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

3.9 Continuity Plate (CP) — A plate provided within the flanges of the column at the levels of the flanges of the beams framing in the direction of the web of the column.

3.10 Demand Critical Weld — A weld connecting two structural components which are part of lateral load resisting system and at least one of which is expected to undergo inelastic straining following yielding.

3.11 Diaphragm — A horizontal or nearly horizontal structural system (for example, reinforced concrete floors and horizontal bracing systems), which transmits lateral forces to vertical elements connected to it.

3.12 Doubler Plate (DP) — A plate provided parallel to the web of the column, and connected along its own perimeter to the web of the column and at some intermediate locations within itself when necessary.

3.13 Eccentrically Braced Frame (EBF) — A lateral load resisting system composed of interconnected beams and columns with inclined members as braces that has at least one end connected to a beam through a link with a defined eccentricity from another beam-to-brace connection, with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resist gravity and lateral force effects primarily by axial action in the braces, and shearing and flexural actions in the links. It is specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

3.14 Gravity Columns — Columns, which are not a part of the lateral load resisting system, and are designed to resist (a) gravity loads, and (b) force effects due to displacement compatibility induced during earthquakes, through axial, flexural and shearing actions.

3.15 Joint Panel Zone (JPZ) — The finite-sized area of the column (from top flange level to bottom flange level of the deepest beam) at the junction where the beams frame into the column.

3.16 Lateral Load Resisting System — The arrangement of structural members which provide resistance to the effects induced by earthquakes.

3.17 Link — The segment of a beam that is located between the ends of the connections of two inclined braces in EBFs. The length of the link is defined as the clear distance between the ends of two diagonal braces.

3.18 Material Strength Uncertainty Factor — The ratio of expected stress (yield or ultimate) to the characteristic stress (yield or ultimate) of a material, greater than unity.

3.19 Moment Resistant Frame (MRF) — A lateral load resisting system composed of interconnected beams and columns, without structural walls and inclined members as braces, which function as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resists gravity and lateral force effects primarily by axial and flexural actions.

3.19.1 *Special Moment Resisting Frame* (*SMRF*) — A MRF specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

3.20 Protected Zone — Area of a member or connection element designed to undergo inelastic straining under design earthquake effects, and is required to be devoid of additional attachments or discontinuities resulting from fabrication and erection procedures.

3.21 Walls — Planar members (generally vertical), generally to be provided to the full height of the building, resisting loads through axial, flexural and shearing actions in its own plane.

4 SYMBOLS

For the purpose of this standard, the following letter symbols shall have the meaning indicated below against each of them:

A_f	Area of flange
A_g	Gross cross-sectional area
A_{gL}	Gross cross-sectional area of link
A_n	Net cross-sectional area
A_{st}	Area of stiffener
A_{wl}	Area of web of link
b	Width of flange plate
b_{bf}	Overall breadth of beam flange
b_{cf}	Overall breadth of column flange
DL	Dead load as per IS 875 (Part 1)
d	Depth of section
d_b	Overall depth of beam
d_c	Depth of column
d_f	Distance between centroids of flanges of a section under bending
d_{pz}	Depth of panel zone
E	Modulus of elasticity of steel = 200 GPa
EL	Earthquake load as per IS 1893 (Part 1)
EL_m	Estimated maximum equivalent earthquake load induced in the structure
e	Length of link
F_i	Lateral load acting on storey i
f_u	Characteristic ultimate tensile stress of structural steel
f_y	Characteristic yield tensile stress of structural steel
f_{yb}	Characteristic yield tensile stress of structural steel used in beam
f_{yc}	Characteristic yield tensile stress of structural steel used in column
H_i	Height of storey i
h	Inter-storey height
I_x	Second moment of area about major axis of bending
I_y	Second moment of area about minor axis of bending
K_{br}	Required shear stiffness of panel bracing system
L_{br}	Unbraced length along the centerline of a member
LL	Live load as per IS 875 (Part 2)
M	Bending moment
M_{bo}	Estimated overstrength moment capacity of beam = $\sum 1.1 R_y Z_{pb} f_{yb}$
M_{pb}	Plastic moment capacity of beam section
M_{pL}	Plastic bending moment capacity of link
M_{pc}	Plastic moment capacity of column section
P	Axial load
$\bar{\rho}$	Normalised axial force parameter considering effect of shear force demand (*see* **11.3**)
P_{br}	Required strength of connection of panel bracing system
P_d	Design compressive strength determined using IS 800
P_u	Maximum factored axial load demand
P_y	Yield axial load capacity
R_u	Ratio of the expected tensile stress to the characteristic tensile stress
R_y	Ratio of the expected yield stress to the characteristic yield stress
r_y	Radius of gyration about the weaker axis of bending
S_h	Factor to account for strain hardening and strain rate in links: 1.25 for I-shaped links and 1.4 for box shaped links

t	Thickness of component plate
t_{bf}	Thickness of beam flange
t_{cf}	Thickness of column flange
t_{pz}	Thickness of joint panel zone including the thickness of doubler plates if provided
V_{br}	Required shear strength of panel bracing system
V_{pL}	Plastic shear capacity of link
V_{pzc}	Nominal shear force capacity of panel zone
V_{pzd}	Shear force demand on panel zone at the face of the column
W_i	Weight acting on storey i
w_{pz}	Width of panel zone
Z_p	Plastic section modulus
Z_{pb}	Plastic section modulus of beam
Z_{pc}	Plastic section modulus of column
Z_{pL}	Plastic section modulus of link
α_i	Ratio of the secondary overturning moment to primary overturning moment at storey i
Δ_i	Lateral displacement of storey i, from linear elastic analysis
γ_{LL}	Partial safety factor for live load
Ω	Overstrength factor of the building
ε	Yield stress ratio = $(250/f_y)^{1/2}$

Unless otherwise specified, all dimensions are in millimetre (mm), loads in Newton (N), stresses in Mega-Pascal (MPa).

5 GENERAL SPECIFICATIONS

5.1 Structural Design

The design and construction of steel buildings shall be governed by provisions of IS 800, except as modified by the provisions of this standard for those members participating in the lateral load resistance under earthquake shaking.

5.2 Materials

The provisions of this standard on earthquake resistant design of structural steel buildings are applicable for the following grades of materials:

a) structural steel sections and plates of grades E250 (B0 or C), E275 (B0 or C), E300 (B0 or C) or E350 (B0 or C) as per IS 2062;

b) any other equivalent grade of steel satisfying the following:

   1) characteristic yield stress f_y of structural steel sections and plates in which inelastic action is expected shall not exceed 350 MPa;

   2) Structural steel shall have minimum elongation of 22 percent;

   3) Structural steel shall have minimum charpy V-notch impact test value of 27 J at 0 °C; and

   4) The ratio of the ultimate strength to the yield strength shall at least be 1.15.

For the purpose of this standard, the expected yield strength and expected ultimate strength of a member, section or connection shall be determined based on the expected yield stress and expected tensile stress. The expected yield stress and expected tensile stress of a grade of steel shall be considered to be equal to $R_y f_y$ and $R_y f_u$ respectively, where R_y is the ratio of the expected yield stress to the characteristic yield stress f_y, and R_u the ratio of the expected tensile stress to the characteristic tensile stress f_u of that material. In short, R_y and R_u are referred as material strength uncertainty factors; their values for different grades of steel are given in Table 1.

5.2.1 For any other steel satisfying **5.2** (b), values of 1.4 and 1.2 shall be considered for R_y and R_u respectively of such steel.

Table 1 Material Strength Uncertainty Factors R_y and R_u
(*Clause* 5.2)

Sl No.	Grade of Steel	Material Strength Uncertainty Factor	
		R_y	R_u
(1)	(2)	(3)	(4)
i)	E250 (B0 or C)	1.4	1.2
ii)	E275 (B0 or C)	1.4	1.2
iii)	E300 (B0 or C)	1.3	1.1
iv)	E350 (B0 or C)	1.2	1.1

5.2.2 All welds, conforming to relevant Indian Standards, shall be made with filler metals satisfying minimum elongation of 20 percent, and charpy V-notch impact test value of 47 J at 0 °C.

5.2.3 All demand critical welds shall have charpy V-notch impact test value of at least 27 J at - 20 °C.

5.2.4 Bolts, nuts and washers shall conform to relevant Indian Standards, such as IS 1363 (Parts 1, 2 and 3), IS 1364 (Part 1 to 6), IS 3640, IS 3757, IS 4000, IS 5624, IS 6610, IS 6639 and IS 6649.

5.3 Section Classification

Structural sections of lateral load resisting system shall comply with the width-to-thickness requirements specified in Table 2.

5.4 Design Requirement

The general requirements for ductile design and detailing of steel buildings shall ensure that required capacity is provided to cater to the imposed demand, in terms of the following broad aspects:

a) Stability
b) Stiffness
c) Strength
d) Deformability and ductility

Here, the words capacity and demand refer to all the aspects specified above. Thus, steel buildings designed and detailed as per this standard are expected to resist design earthquake hazard defined in IS 1893 (Part 1) without collapse.

5.5 Loads and Load Combinations

Design earthquake loads (*EL*) shall be estimated and combined as per IS 1893 (Part 1), along with those in Table 4 of IS 800. In addition, the following load combinations shall be considered for design of: (a) columns in SMRFs, SCBFs and EBFs, (b) beams in SCBFs and EBFs, (c) braces in EBFs, and (d) all connections which are part of structural system in steel buildings:

$$1.2\,DL + \gamma_{LL} LL \pm 1.0\,EL_m \qquad \text{.......(1)}$$

$$0.9\,DL \pm 1.0\,EL_m \qquad \text{......(2)}$$

where

DL = Dead load as per IS 875 (Part 1);

γ_{LL} = Partial safety factor for live load

$= \begin{cases} 0.25 \text{ for live load class less than or equal to } 3.0 \text{ kN/m}^2 \\ 0.50 \text{ for live load class more than } 3.0 \text{ kN/m}^2; \end{cases}$

LL = Imposed load as per IS 875 (Part 2);

EL_m = Estimated maximum equivalent earthquake force induced in the structure = $\Omega\ EL$;

Ω = Overstrength factor = 2.5 for SCBFs and EBFs = 3.0 for SMRFs; and

EL = Earthquake load as per IS 1893 (Part 1).

5.6 Analysis

Structural analysis of buildings, under the action of design load combinations, shall be performed as outlined in **4.4** of IS 800, unless specifically altered for the purpose of complying with the requirements of this standard.

5.7 Stability

Under the action of the design loads, a building and its components shall be stable, and overall force and moment equilibrium shall be satisfied. Thus, the building shall not slide or overturn under the action of the design loads specified in **5.5**.

5.7.1 *Stability Bracing Requirement for Members*

When required for structural systems defined in **12**, stability bracing shall be provided as specified to restrain flexural or lateral-torsional buckling of steel components or members subject to axial compression, bending moment or shear force. The strength of bracing connections shall be at least 1.5 times the corresponding strength of the bracing.

Table 2 Limiting Width-to-Thickness Ratios for Compression Elements of Earthquake Resistant Structures

(*Clauses* 5.3, 6.1, 7.1, 10.1, 11.1 *and* 12.2.4.1)

Sl No.	Component	Section Type	Limiting Plate Slenderness	
			Outstanding Flange Width-to-Thickness Ratio	Web Depth-to-Thickness Ratio
(1)	(2)	(3)	(4)	(5)
i)	Beam	Doubly-symmetric rolled I-sections	$\frac{9.0\varepsilon}{\sqrt{R_y}}$	$\frac{44.5\varepsilon}{\sqrt{R_y}}$
		Doubly-symmetric built-up I-sections		
ii)	Column	Doubly-symmetric rolled I-sections	$\frac{9.0\varepsilon}{\sqrt{R_y}}$	$\frac{72.7\varepsilon}{\sqrt{R_y}}(1-1.04C_a)$ for $C_a \leq 0.118$ and $\frac{24.9\varepsilon}{\sqrt{R_y}}(2.68-C_a) \geq \frac{44.4\varepsilon}{\sqrt{R_y}}$ for $C_a > 0.118$
		Doubly-symmetric built-up I-sections		
iii)	Brace	Rolled or built-up I-sections	$\frac{11.3\varepsilon}{\sqrt{R_y}}$	$\frac{44.4\varepsilon}{\sqrt{R_y}}$
		Closed box sections	$\frac{21.4\varepsilon}{\sqrt{R_y}}$ (Flange width is the flange width minus thickness of the webs)	$\frac{21.4\varepsilon}{\sqrt{R_y}}$
iv)	Links	Doubly-symmetric rolled or built-up I-sections	$\frac{11.3\varepsilon}{\sqrt{R_y}}$	$\frac{44.4\varepsilon}{\sqrt{R_y}}$
		Closed box sections	$\frac{21.4\varepsilon}{\sqrt{R_y}}$ (Flange width is the flange width minus thickness of the webs)	$\frac{49.4\varepsilon}{\sqrt{R_y}}$

where

$\varepsilon = (250/f_y)^{1/2}$; and

$C_a = P_u/(P_y/\gamma_{mo})$

5.8 Stiffness

Buildings shall have adequate lateral stiffness to satisfy the drift limit specified in IS 1893 (Part 1), and structural members shall satisfy deflection limits specified in IS 800, unless specified otherwise in this standard.

5.8.1 *Drift*

The storey-drift for SMRFs shall be estimated considering the stiffness of the finite sized beam-column joints. In the absence of rigorous analysis, centreline dimensions of members shall be used to estimate storey drift. For braced frames too, centreline dimensions of members shall be used.

5.8.2 *P-Δ Effect*

For moment resisting frames in particular, detailed geometric non-linear analysis shall be carried out. But, the same may be neglected for any storey *i*, when the ratio α_{i} of the secondary moment to the primary moment complies with:

$$\alpha_{\mathrm{i}} = \frac{W_{\mathrm{i}} \Delta_{\mathrm{i}}}{H_{\mathrm{i}} F_{\mathrm{i}}} \leq 0.04$$

A value of $\alpha_{\mathrm{i}} > 0.1$ is not permitted.

5.9 Strength

Buildings, structural members and components shall have adequate strength as required by IS 1893 (Part 1), IS 800, and as required by this standard.

5.9.1 *Capacity Design*

Capacity design principles shall be adopted in design of structural components, members and buildings to resist earthquake shaking as specified in this standard.

5.10 Deformability and Ductility

Buildings shall be considered to satisfy the requirements of deformability and ductility by conforming to the requirements of this standard.

6 BEAMS

6.1 Sections

Only doubly-symmetric parallel flange standard rolled sections (as per IS 808) or built-up sections, with flange width to thickness ratio and web depth to thickness ratio values less than the limits specified in Table 2, shall be used as beams. The flange to web weld in built-up beams shall be continuous.

6.2 Slenderness

The ratio of the maximum unbraced length of the compression flange of a beam L_{br} to the radius of gyration r_{y} about the weaker axis of the beam cross-section shall not exceed 25 for a distance of $2d_{\mathrm{b}}$ from the end of the beam to column connection. $L_{\mathrm{br}}/r_{\mathrm{y}}$ for the remaining portion of the beam shall not exceed $0.10E/(R_{\mathrm{y}} f_{\mathrm{yb}})$.

6.3 Bracing

The stiffness of bracing shall be as given below:

a) Beams shall be restrained against rotation about their longitudinal axis at supports and at intermediate locations along the length of the beam through the use of internal panel bracing without any external rigid support.

b) The lateral bracing shall be attached at or near the compression flange of the beam.

c) The lateral bracing shall be attached at or near both flanges, near the point of inflection in beams bending in double curvature.

6.3.1 *Stiffness of Bracing*

The stiffness of bracing shall be as given below:

a) The shear stiffness of the panel bracing system closest to the inflection point in a beam bending in double curvature shall be:

$$K_{\mathrm{br}} \geq \frac{10 R_{\mathrm{y}} M_{\mathrm{pb}}}{L_{\mathrm{br}} d_{\mathrm{f}}}$$

where L_{br} is the unbraced length; and d_{f} is the distance between centroids of the flanges of the beam.

b) The shear stiffness of the panel bracing system other than near the inflection point in a beam bending in double curvature shall be:

$$K_{\mathrm{br}} \geq \frac{5 R_{\mathrm{y}} M_{\mathrm{pb}}}{L_{\mathrm{br}} d_{\mathrm{f}}}$$

6.3.2 *Strength of Bracing*

The shear strength of the panel bracing system shall be:

$$V_{\mathrm{br}} \geq \frac{0.025 R_{\mathrm{y}} M_{\mathrm{pb}}}{d_{\mathrm{f}}}$$

6.3.3 *Special Bracing at Plastic Hinge Locations*

Special bracing shall be located adjacent to the expected plastic hinge locations. Both flanges of beams shall be laterally braced. The axial strength of such lateral bracing shall be:

$$P_{\mathrm{br}} \geq \frac{0.06 R_{\mathrm{y}} M_{\mathrm{pb}}}{d_{\mathrm{f}}}$$

and the required bracing stiffness shall be as in **6.3.1** (b).

6.4 Strength

The design strength of beam shall satisfy the load combinations in IS 1893 (Part 1), and the overstrength load combinations specified in **5.5** in SCBFs and EBFs, or when a beam is part of diaphragm collector or chord.

6.4.1 *Shear Strength*

The design shear strength of the beam at the location of the plastic hinge shall be determined as per IS 800, and it shall be at least equal to the shear demand specified in **8.4**.

6.5 Splice

Beam splices shall be located at least $3d_{\mathrm{b}}$ awayfrom the face of the column or d_{b} from the line of action of any concentrated force acting on the beam. The design strength of beam splices shall at least be 1.80 times the required strength, except at beam-column connections. Further, design strength of each flange splice plate shall at least be $1.2R_{\mathrm{y}} f_{\mathrm{y}} A_{\mathrm{f}}$, where A_{f} is the area of the flange being spliced.

7 COLUMNS

7.1 Sections

Only doubly-symmetric parallel flange standard rolled as per IS 808 or built-up sections, with flange width to thickness ratio, and web depth to thickness ratio less than the corresponding limits specified in Table 2, shall be used as columns. The flange to web weld in built-up beams shall be continuous.

7.2 Slenderness

The slenderness ratio of unbraced length of columns shall be less than 75.

7.3 Bracing

Columns shall be laterally braced at supports and at intermediate locations along the length of the column through the use of internal panel bracing without any external rigid support.

7.3.1 *Stiffness of Bracing*

The shear stiffness of the panel bracing system, in the direction perpendicular to the longitudinal axis of the columns shall be:

$$K_{\mathrm{br}} \geq \frac{3P_{\mathrm{u}}}{L_{\mathrm{br}}}$$

where

P_{u} = Maximum factored axial load; and
L_{br} = Unbraced length of the column.

7.3.2 *Strength of Bracing*

The shear strength of the panel bracing system shall at least be:

$$V_{\mathrm{br}} \geq 0.005 P_{\mathrm{u}}$$

7.3.3 *Strength of Bracing Connection*

The connection of the bracing system to the column shall have design strength at least equal to:

$$P_{\mathrm{br}} \geq 0.01 P_{\mathrm{u}}$$

subject to minimum design action on connection given in IS 800.

7.4 Strength

Columns shall have design strength more than the maximum demand arising from the following:

a) Structural analysis based on load combinations specified in **5.5**, and

b) Maximum loads transferred to the column considering 1.2 times the strength of the connected members (beams, braces, etc) determined considering material strength uncertainty factor.

7.4.1 Columns in both moment frames and braced frames that are common to intersecting frames aligned along two orthogonal directions, shall consider in design the potential for simultaneous inelasticity from all such frames for determination of the required axial strength, including the overstrength earthquake load or the capacity-limited earthquake load, as applicable.

The direction of application of the load in each such frame shall be selected to produce the most severe load effect on the column.

7.4.2 Columns in buildings designed to resist lateral loads shall not carry tensile forces.

7.5 Splice

Column splices shall be located in the middle third of the height of the columns, at least 1.0 m away from the beam-to-column moment connection.

The design strength demand of column splices shall at least be that determined using **5.5**. Further, the design strength of both flange and web splice plates shall at least be $1.2R_y$ times of their respective strengths.

8 BEAM-COLUMN JOINT

At a beam–column joint, the following design aspects shall be addressed:

a) Column to beam strength ratio,
b) Joint panel zone design, and
c) Beam–column connection design.

8.1 Basis of Design

Flexural plastic hinges are expected to be formed at the end regions of the beams away from the column face. Under this condition, the column and the beam-column joint, including the beam-column connection, is expected to remain elastic and shall be designed as capacity protected elements.

8.2 Column to Beam Strength Ratio

At a beam–column joint, the following strength ratio shall be satisfied:

$$\frac{\sum M_{pc}}{\sum M_{bo}} = \frac{\sum Z_{pc} f_{yc} \left(1 - \frac{P_u}{P_d}\right)}{\sum 1.1 R_y Z_{pb} f_{yb}} > 1.4$$

where Z_{pc} and Z_{pb} are the plastic section modulus and f_{yc} and f_{yb} are the characteristic yield strength of column and beam cross-sections respectively, P_u is the maximum factored axial compressive load and P_d is the design strength under axial compression, and R_y is the material uncertainty factor corresponding to the grade of steel in beams.

8.2.1 The above requirement need not be satisfied at the roof level.

8.3 Joint Panel Zone

Shear yielding of joint panel zone (JPZ) shall be limited. Use of continuity and doubler plates is permitted (Fig. 1).

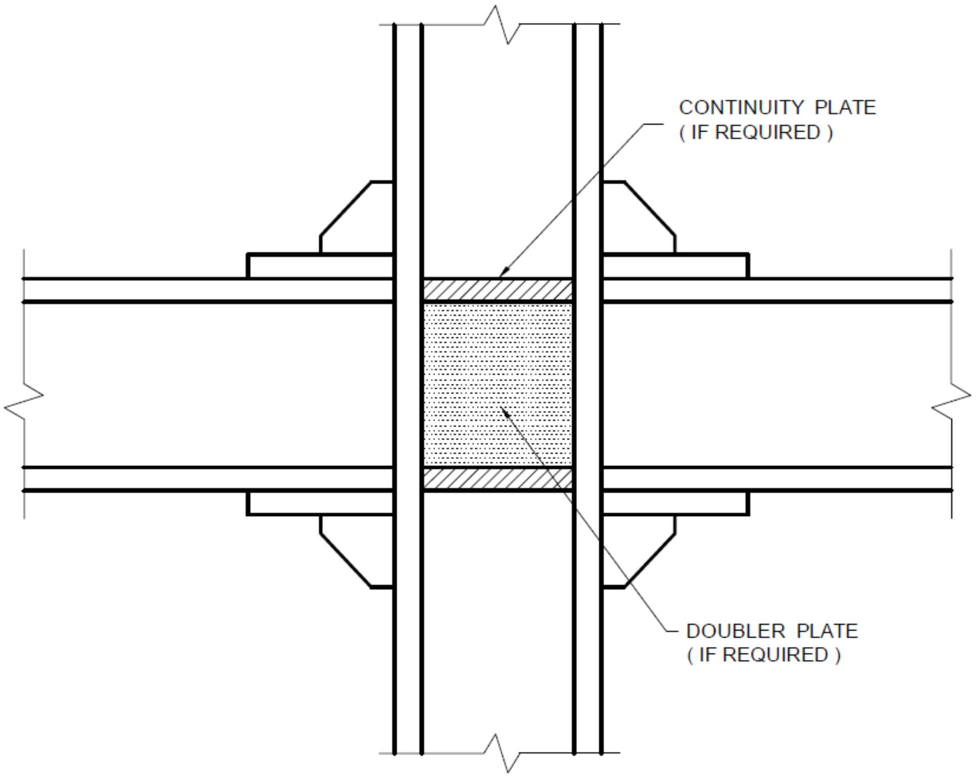


FIG. 1 TYPICAL INTERIOR REINFORCED BEAM-COLUMN JOINT

8.3.1 *Panel Zone Demand*

The shear force demand at the face of the column flanges shall be taken as:

$$V_{\mathrm{pzd}} = \sum \left(\frac{1.1 R_{\mathrm{y}} f_{\mathrm{y}} Z_{\mathrm{pb}}}{0.95 d_{\mathrm{b}}} \right)$$

8.3.2 *Panel Zone Capacity*

The design shear strength capacity shall be:

$$\mathrm{V_{pzc}} = \frac{f_{\mathrm{y}}}{\sqrt{3}\gamma_{\mathrm{m0}}} \, (0.95\mathrm{d_c}) t_{\mathrm{pz}}$$

where

t_{pz} = thickness of the panel zone, including thickness of doubler plates if provided; and

γ_{m0} = 1.1.

8.3.3 *Panel Zone Thickness*

The individual thickness of the column web and doubler plates (when provided), shall be more than:

$$\frac{d_{\mathrm{pz}} + w_{\mathrm{pz}}}{90}$$

where

$d_{\mathrm{pz}} = d_{\mathrm{b}} - 2t_{\mathrm{bf}}$ of the deeper beam, in mm; and

$w_{\mathrm{pz}} = d_{\mathrm{c}} - 2t_{\mathrm{cf}}$, in mm.

8.3.4 *Doubler Plate*

Doubler plates with plug welding shall be provided when the thickness of the column web within the panel does not satisfy the strength requirements. It is permitted to use doubler plates with or without continuity plates. When continuity plates are not provided, doubler plates shall extend at least 150 mm beyond the deeper beam flange levels on either sides of the panel zone.

8.3.5 *Continuity Plate*

Continuity plates shall be provided when:

$$\frac{6.25 \, f_{\mathrm{y}} {t_{\mathrm{cf}}}^2}{1.1} \leq 1.2 R_{\mathrm{y}} f_{\mathrm{y}} b_{\mathrm{bf}} t_{\mathrm{bf}}$$

When provided, thickness of continuity plate shall not be less than thickness of the thinner beam flange on either side of the column, and width not less than the distance of the tip of the wide beam flange from the face of the column web. If different grades of steel are used, appropriate values of f_{y} shall be used on either side of the inequality.

8.4 Beam-Column Connection

Fully-restrained, reinforced beam-column connections shall be used in moment frames, capable of transferring at least a bending moment of $1.1 R_{\mathrm{y}} f_{\mathrm{y}} Z_{\mathrm{pb}}$, and shear demand determined based on capacity design principle considering, (a) beams bending in double curvature, (b) plastic hinges of strength $1.1 R_{\mathrm{y}} f_{\mathrm{y}} Z_{\mathrm{pb}}$ assumed to act at a distance $d_{\mathrm{b}}/2$ from the end of the connection, and (c) gravity load required to be carried.

8.4.1 *Welded Beam-Column Connection*

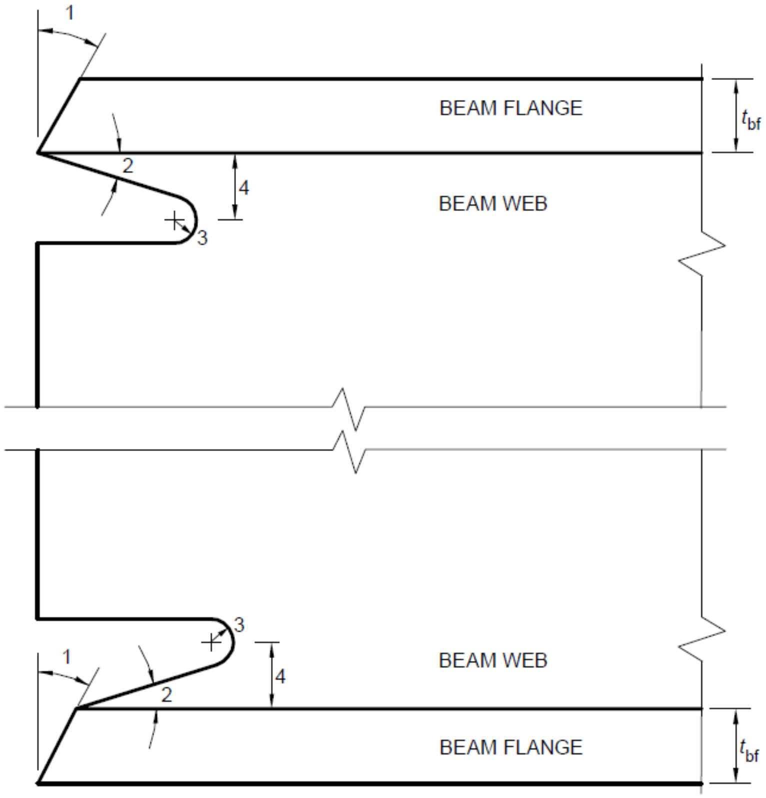
In general, beam flanges shall be connected to column flanges using complete joint penetration groove welds, while beam web shall be connected to the column flange using either a complete joint penetration groove weld extending between weld access holes, or using a bolted single plate shear connection.

A weld access hole detail shall be adopted to ensure that the location of maximum plastic strain does not occur at the interface between the beam web and beam flange but is entirely in the beam web (Fig. 2). Use of no weld access hole detail is not permitted. Removal of backing bar after welding and finishing of surface through grinding shall be ensured.

In general, a cover plated welded flange connection with welded rib plates at both beam flange levels is preferable.

9 COLUMN BASE

Column bases may have any form of embedded connection or anchor bolted base plate connection. The degree of fixity offered by a connection should be established and used in structural analysis.



NOTE — Typical details for rolled beam shapes welded from one side steel backing plates:
1) Standard bevel;
2) Angle not to exceed 25°;
3) Radius not less than 10 mm; and
4) Dimension not less than ¾ t_{bf} or 20 mm, whichever is greater.

FIG. 2 WELD ACCESS HOLE

9.1 Anchor Bolted Base Plate Connection

Anchor bolted base plate connections at column bases shall be designed to prevent the following:

a) Bearing failure of concrete under compression,
b) Pullout cone failure of concrete due to tensile force in anchor bolts,
c) Side face blowout failure of concrete due to tensile force in anchor bolts with headed or hooked ends,
d) Wedge-cone failure of concrete due to shear force in anchor bolts,
e) Pryout cone failure of concrete due to shear force in anchor bolts, and
f) Bolt-concrete bond slip failure.

9.2 Strength

The required design strength of the steel elements at the column base, including base plate, anchor bolts, stiffening plates, and shear lug elements shall be determined for the load combinations given in **5.5**.

9.3 Fixed Column Base

Fixed column base connections and supporting foundation shall be designed to resist moment demand of $1.1R_{\mathrm{y}}M_{\mathrm{pc}}$ and shear demand equal to $2.2R_{\mathrm{y}}\,M_{\mathrm{pc}}/H_{\mathrm{c}}$, where M_{pc} and H_{c} are the plastic moment capacity and the clear height of the column between the connections, respectively.

9.4 Pinned Column Base

In lieu of detailed calculations establishing rotational stiffness (based on the degree of fixity) and bending moment strength characteristics, it is permitted to analyse and design anchor bolted base plate connections at column bases in buildings as pinned connection. In such cases, the connection and supporting foundation shall be designed for minimum moment of $0.5R_{\mathrm{y}}M_{\mathrm{yc}}$, where M_{yc} is the yield moment capacity of the column section, in addition to shear force demand equal to $1.1 R_y M_{pc}/H_c$.

10 STRUCTURAL BRACES

Structural braces may be used to impart lateral stiffness and strength to building frames. Such braces shall be provided in selected bays over the full height of the building frame. Braces part of concentrically braced frames (CBF) shall also conform to the requirements given hereunder.

10.1 Sections

Standard rolled or built-up sections or closed box sections, with flange width to thickness ratio and web depth to thickness ratio less than the limits specified in Table 2, shall be used as braces. The weld between the elements of built up section shall be continuous.

10.2 Slenderness

The effective slenderness ratio of braces shall be less than 160. In case of built-up braces, at least two connectors shall be provided at uniform spacing such that the slenderness ratio of individual plate elements between the connectors shall be less than 0.4 times the governing effective slenderness ratio of the built-up brace.

The sum of shear strengths of the connectors shall be greater than the expected tensile strength of each element, except when buckling of the brace about its critical buckling axis does not cause shear in the connectors. Connectors shall not be located within the middle one-fourth of the clear brace length.

10.3 Effective Area

Brace effective net area shall not be taken less than the gross cross-sectional area of the brace. Where reinforcement on braces is used, the following requirements shall apply:

a) The characteristic yield strength of the reinforcement, when provided in the form of steel plates, shall be at least equal to the characteristic yield strength of the brace, and

b) The connections of the reinforcement to the brace shall have sufficient strength to develop the expected reinforcement strength.

10.4 Bracing Connection

The required strength in tension, compression, and flexure of brace connections (including beam-to-column connections if part of the braced-frame system) shall be determined as required in the following. These required strengths are permitted to be considered independently without interaction.

10.4.1 *Tensile Strength*

The tensile strength of brace connections shall at least be the lesser of the following:

a) The expected yield strength in tension of the brace, determined as maximum of $1.1 R_y f_y A_g$ and $R_u f_u A_n$; and
b) The maximum load effect, indicated by analysis as in **5.6**, that can be transferred to the brace by the system.

10.4.2 *Compressive Strength*

The compressive strength of brace connections shall at least be equal to the brace strength in compression, generally as governed by buckling.

10.4.3 *Accommodation of Brace Buckling*

Brace connections shall be designed to withstand the flexural forces and rotations imposed by brace buckling. Connections satisfying the following provisions are deemed to satisfy this requirement:

a) Required Flexural Strength

Brace connections designed to withstand the flexural forces imposed by brace buckling shall have a required flexural strength equal to 1.1 times the expected brace flexural strength. The expected brace flexural strength shall be determined as $R_y f_y Z_p$ of the brace about the critical buckling axis.

b) Rotation Capacity

Brace connections shall be designed to withstand the rotations imposed by brace buckling. Inelastic rotation of the connection is permitted.

10.4.4 *Gusset Plates*

To accommodate brace buckling, gusset plates shall be detailed to undergo out-of-plane bending and welds that attach a gusset plate directly to a beam flange or column flange shall be designed to have shear strength per unit length equal to $\frac{R_y f_y t_p}{\sqrt{3}}$, where t_p is the thickness of the gusset plate.

11 SHEAR LINKS

Shear links may be used as structural fuse in eccentrically braced frames (EBF) as specified in this standard. These links are expected to be subjected to combined action of bending moment and shear force and undergo yielding under earthquake effects.

11.1 Sections

Links shall be I-shaped cross sections (standard rolled wide-flange sections or built-up sections), or built-up box sections, with flange width to thickness ratio and web depth to thickness ratio less than the limits specified in Table 2. Hollow structural steel (HSS) sections shall not be used as links. Further:

a) The web or webs of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted;
b) For links made of built-up cross sections, complete-joint-penetration groove welds shall be used to connect the web (or webs) to the flanges; and
c) Links of built-up box sections shall have a moment of inertia, I_y, about an axis in the plane of the EBF greater than $0.67 I_x$, where I_x is the moment of inertia of the link about an axis perpendicular to the plane of the EBF.

11.2 Link Shear Strength

The design shear strength of link shall be the lower value of the following:

a) For shear yielding: $\dfrac{V_{pL}}{\gamma_{m0}}$

where

$$V_{pL} = \begin{cases} \dfrac{f_y A_{wL}}{\sqrt{3}} & \text{for } P_u/P_y \leq 0.15 \\ \dfrac{f_y A_{wL}}{\sqrt{3}} \sqrt{1 - \left(\dfrac{P_u}{P_y}\right)^2} & \text{for } P_u/P_y > 0.15 \end{cases};$$

$$A_{wL} = \begin{cases} (d_L - 2t_f)t_w & \text{for I-shaped link sections,} \\ 2(d_L - 2t_f)t_w & \text{for box link sections;} \end{cases}$$

P_u = Factored axial load in the link;
P_y = $f_y A_{gL}$;
A_{gL} = Gross cross-sectional area of link;
d_L = Overall depth of link;
t_f = Thickness of flange;
t_w = Thickness of web; and
γ_{mo} = 1.1

b) For flexural yielding: $\dfrac{2M_{pL}}{e\gamma_{m0}}$

where

$$M_{pL} = \begin{cases} f_y Z_{pL} & \text{for } P_u/P_y \leq 0.15 \\ f_y Z_{pL}\left[1 - \left(\dfrac{P_u}{P_y}\right)\right] & \text{for } P_u/P_y > 0.15 \end{cases};$$

and

Z_{pL} = Plastic section modulus of link about the bending axis.

11.3 Length of link

The length of link *e* shall be less than $1.6\, M_{pL}/V_{pL}$, where M_{pL} and V_{pL} are the plastic bending moment capacity and plastic shear capacity of the link, as per **11.2**. Further, the following shall be satisfied:

If $P_u/P_y > 0.15$, the length of the link shall be limited as follows:

$$e \leq \begin{cases} 1.6\, M_{pL}/V_{pL} & \text{for } \bar{\rho} \leq 0.5 \\ 1.6\, M_{pL}(1.15 - 0.3\bar{\rho})/V_{pL} & \text{for } \bar{\rho} > 0.5 \end{cases}$$

where

$\bar{\rho} = \dfrac{P_u/P_y}{V_u/V_y}$, and

$V_y = \dfrac{f_y A_{wL}}{\sqrt{3}}$.

11.4 Stiffeners for I-Shaped Link Sections

Web of links shall be stiffened to prevent premature buckling. The required strength of fillet welds connecting a stiffener to the link web shall be $f_y A_{st}$, where A_{st} is the horizontal cross-sectional area of the link stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $\dfrac{f_y A_{st}}{4}$.

11.4.1 *End Web Stiffeners*

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than the larger of $0.75 t_w$ or 10 mm, where b_f and t_w are the link flange width and link web thickness, respectively.

11.4.2 *Intermediate Web Stiffeners*

Links shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_{\mathrm{w}} - 0.2d)$.

11.5 Stiffeners for Box Link Sections

Web of links shall be stiffened to prevent premature buckling. The required strength of fillet welds connecting a stiffener to the link web shall be $f_{\mathrm{y}}A_{\mathrm{st}}$, where A_{st} is the horizontal cross-sectional area of the link stiffener.

11.5.1 *End Web Stiffeners*

Full-depth web stiffeners shall be provided on one side of each link web at the diagonal brace connection. These stiffeners are permitted to be welded to the outside or inside face of the link webs. These stiffeners shall each have a width not less than $\frac{b}{2}$, where b is the inside width of the box section. These stiffeners shall each have a thickness not less than the larger of $0.75t_{\mathrm{w}}$ or 10 mm.

11.5.2 *Intermediate Web Stiffeners*

Box links shall be provided with full-depth intermediate web stiffeners welded either to the outside or inside face of the link webs as follows:

a) When web depth-to-thickness ratio is greater than $19\epsilon/\sqrt{f_{\mathrm{y}}}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20t_{\mathrm{w}} - (d - 2t_{\mathrm{f}})/8$; and
b) When web depth-to-thickness ratio is less than or equal to $19\epsilon/\sqrt{f_{\mathrm{y}}}$, no intermediate web stiffeners are required.

12 SPECIAL REQUIREMENTS FOR STRUCTURAL SYSTEMS

Special requirements of special moment resisting frame (SMRF), special concentrically braced frame (SCBF), and eccentrically braced frame (EBF), designed as lateral load resisting system in buildings, are provided hereunder.

12.1 Special Moment Resisting Frames

Special moment resisting frames (SMRFs) of structural steel shall be designed to satisfy the requirements of this section.

12.1.1 *Basis of Design*

Special moment resisting frames designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the beams, limited yielding of panel zones, and little or no yielding of columns except at base. Special moment resisting frames may be used in any seismic zones except in buildings taller than 15 m in seismic zones IV and V [*see* IS 1893 (Part 1)] and for any buildings (importance factor values). Yielding of beam to column connections in SMRFs shall not be permitted by this standard.

12.1.2 *Analysis*

It is preferable to plan buildings to have independent planar lateral load resisting moment frames in each principal plan directions. In such cases, there are no special analysis requirements. But when two moment frames oriented in orthogonal directions intersect, the possibility of beam yielding in both orthogonal directions simultaneously shall be considered in the design of the common column.

12.1.3 *System Requirements*

The requirements given hereunder shall be satisfied by the building system.

12.1.3.1 *Beams*

Beams in SMRFs are permitted to carry gravity loads through composite action with reinforced concrete slab. For lateral load action, composite action shall not be considered. Further, abrupt changes in beam flanges, through actions like drilling of holes or trimming of flange width, and use of shear studs are prohibited in the beam end regions of length at least twice the depth of the beam where flexural plastic hinges are expected to be formed.

12.1.3.2 *Beam column connections*

Beam to column connections shall be capable of accommodating storey drift angle of 0.04 radians, without loss of strength exceeding 15 percent of the beam plastic moment capacity.

12.1.3.3 *Column to beam strength ratio*

The requirements specified in **8.2** shall apply.

12.1.4 *Member Requirements*

The requirements given hereunder shall be satisfied by the building component or member.

12.1.4.1 *Beams*

Requirements specified in **6** shall apply.

12.1.4.2 *Columns*

Requirements specified in **7** shall apply.

12.1.4.3 *Joint panel zone*

Requirements specified in **8.3** shall apply.

12.1.4.4 *Beam column connections*

Requirements specified in **8.4** shall apply.

12.1.4.5 *Protected zones*

The region at each end of a beam equal to twice the depth of the beam subjected to inelastic straining shall be designated as a protected zone. Steel headed stud anchors and other fabrication and erection attachments shall not be placed on beam flanges within the protected zone.

12.1.4.6 *Column splices*

Requirements specified in **7.5** shall apply.

12.1.4.7 *Demand critical welds*

The following welds shall be designed as demand critical welds:

a) Groove welds at column splices;
b) Welds at column-to-base plate connections, except when:
   1) column plastic hinge at, or near the base plate is precluded by conditions of restraint, or
   2) there is no net tension under load combinations including the overstrength earthquake load; and
c) Welds in beam-column connections.

12.2 Special Concentrically Braced Frames

Special concentrically braced frames (SCBF) of structural steel shall be designed to satisfy the requirements given hereunder. Collector beams that connect SCBF braces shall be considered to be part of SCBF.

12.2.1 *Basis of Design*

In SCBFs, members shall be concentrically connected. But eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design, and the eccentricities do not change the expected source of inelastic deformation capacity of the building. SCBF designed in accordance with these provisions are expected to provide inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.

12.2.2 *Analysis*

The following shall be satisfied in the analysis of SCBFs:

12.2.2.1 The required strength of braces shall be determined based on the analysis required by IS 1893 (Part 1). Further, the required strength of braces shall not exceed the design strength in axial compression as per **7.1.2** of IS 800.

12.2.2.2 The required strength of capacity-protected elements (columns, beams, struts, collectors and connections) shall be taken as the larger force determined from the following analysis:

a) An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension;

b) An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength; and

c) For multi-tiered braced frames, analyses representing progressive yielding and buckling of the braces from weakest tier to strongest. Analyses shall consider both directions of frame loading.

The expected tensile strength (T_e) of braces in tension shall be taken as:

$$T_e = R_y f_y A_g$$

The expected compressive strength (P_e) of the braces shall be taken as:

$$P_e = R_y \gamma_{m0} P_d$$

where P_d is the design compressive strength as determined using **7.1.2** of IS 800. The expected post-buckling strength of the braces in compression shall be taken as 0.2 times the expected compressive strength (P_e) and the bracing connections shall be assumed to remain elastic.

12.2.3 *System Requirements*

The following system requirements shall be satisfied (*see* Fig. 3).

12.2.3.1 *Diagonal and X-braced frames*

Diagonal and X-braced frames are permitted to be used in SCBF.

12.2.3.2 *V- and inverted V-braced frame*

V- and inverted V-braced frames are permitted to be used in SCBF. In such systems, the beams that are intersected by braces away from beam-to-column connections shall satisfy the following requirements:

a) Beams shall be continuous between columns and adequately braced to prevent lateral torsional buckling; and

b) As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) braced frames, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

12.2.3.3 *Continuity of load path*

For the purpose of this standard, a line of braces is defined as a single line, or parallel lines with a plan offset of 10 percent or less of the building dimension perpendicular to the line of braces interconnected adequately through rigid diaphragm. A diaphragm shall be considered to be rigid if the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is less than 1.2 times the average displacement of the entire diaphragm.

12.2.3.4 *Lateral force distribution*

Braces shall be provided in alternate directions along each line of braces. Along any line of bracing, braces shall be provided such that for lateral loading in either direction, tension braces resist between 30 percent to 70 percent of the total horizontal load.

12.2.3.5 *Multi-tiered braced frames*

A special concentrically braced frame is permitted to be configured as a multi-tiered braced frame (MT-SCBF) when the following requirements are satisfied:

a) Braces shall be used in opposing pairs at every tier level.

b) Struts shall satisfy the following requirements:

   1) Horizontal struts shall be provided at every tier level;

   2) Struts that are intersected by braces away from strut-to-column connections shall also meet the requirements stated in **12.2.2.2** (b). When brace buckling occurs out-of-plane, torsional moments arising from brace buckling shall be considered when verifying lateral bracing or minimum out-of-plane strength and stiffness requirements. The torsional moments shall correspond to $1.1R_yM_p$ of the brace about the critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connection, where M_p is the plastic bending moment.

c) Columns shall satisfy the following requirements:

   1) Columns shall be torsionally braced at every strut-to-column connection location.

   2) Columns shall have sufficient strength to resist forces arising from brace buckling. These forces shall correspond to $1.1R_yM_p$ of the brace about the critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connections.

   3) For all load combinations, columns subjected to axial compression shall be designed to resist bending moments due to second-order and geometric imperfection effects. As a minimum, imperfection effects are permitted to be represented by an out-of-plane horizontal notional load applied at every tier level and equal to 0.006 times the vertical load contributed by the compression brace intersecting the column at the tier level.

   4) Lateral drift in each tier in a multi-tiered concentrically braced frame shall not exceed 0.4 percent of the tier height.

12.2.3.6 *K-braced frames*

K-braced frames shall not be used in SCBF.

12.2.4 *Member Requirements*

The requirements specified hereunder shall be satisfied by the component or member.

12.2.4.1 *Sections*

Columns, beams, braces and struts in multi-tiered concentrically braced in shall comply with the width-to-thickness requirements specified in Table 2.

12.2.4.2 *Braces*

Requirements specified in **10** shall apply.

12.2.4.3 *Protected zones*

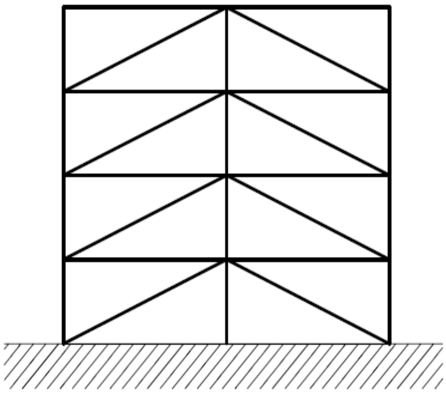
The protected zone of SCBF shall satisfy **5.4**, and shall include the following:

a) For braces, the centre one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling; and

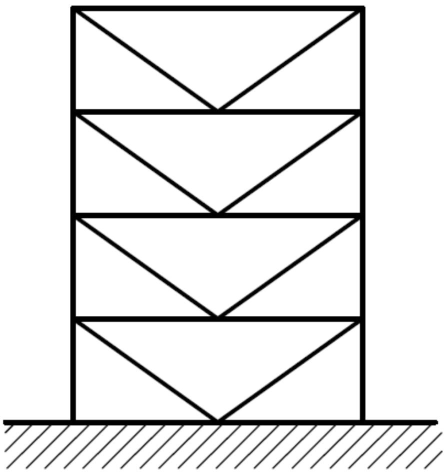
b) Elements that connect braces to beams and columns.

12.2.4.4 *Beams*

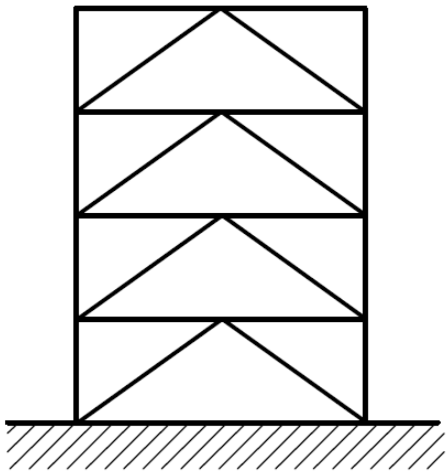
In addition to satisfying requirements of **6**, beams shall be checked for axial load arising due to analysis case **12.2.2.2** (b).



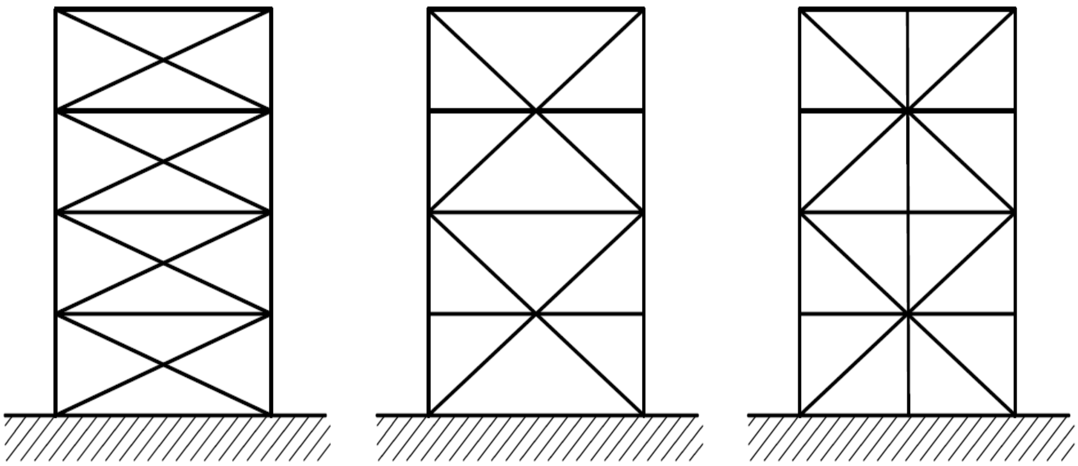
3A SINGLE DIAGONAL BRACED FRAME



3B V-BRACED FRAME



3C INVERTED V-BRACED FRAME



3D X-BRACED FRAMES

FIG. 3 CONCENTRICALLY BRACED FRAMES

12.2.4.5 *Beam-to-column connections*

Where a brace-gusset plate assembly connects to both members at a beam-to-column connection, the connection assembly shall be designed to resist beam moment taken equal to $1.1R_{\mathrm{y}}f_{\mathrm{yb}}Z_{\mathrm{pb}}$. Also, the sum of the expected column flexural strengths shall exceed $1.1R_{\mathrm{y}}f_{\mathrm{yb}}Z_{\mathrm{pb}}$.

This connection moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the over-strength earthquake load.

12.2.4.6 *Column splices*

Requirements specified in **7.5** shall apply. In addition, column splices shall be designed to develop at least 50 percent of the plastic flexural strength, M_{p}, of the smaller connected member, and have shear strength greater than $\sum M_{\mathrm{p}}/H_{\mathrm{c}}$:

where

$\sum M_{\mathrm{p}}$ = Sum of the plastic flexural strengths, at the top and bottom ends of the column; and

H_{c} = Clear height of the column between beam connections.

12.2.4.7 *Demand critical welds*

See **12.1.4.7**.

12.3 Eccentrically Braced Frame

Eccentrically braced frames (EBF) of structural steel shall be designed to satisfy the requirements given hereunder.

12.3.1 *Basis of Design*

These provisions are applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centrelines of the beam and an adjacent brace or column, forming a link that is, subject to shear and flexure (*see* Fig. 4).

Eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear (or flexural) yielding in the links. Links shall not be connected directly to columns.

12.3.2 *Analysis*

The requirements specified hereunder shall be satisfied in the analysis of EBFs.

12.3.2.1 The shear demand on the links shall be determined based on the analysis required by IS 1893 (Part 1).

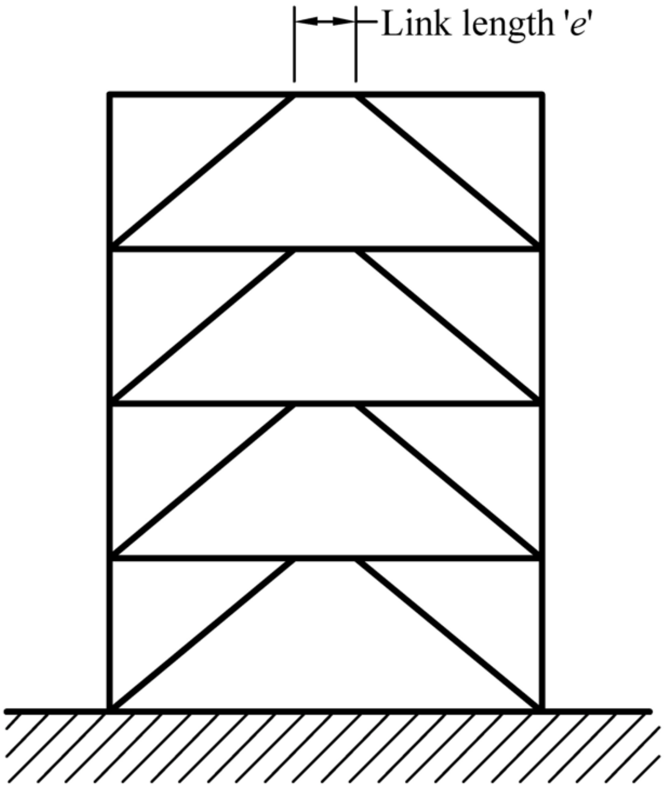


FIG. 4 ECCENTRICALLY BRACED FRAME

12.3.2.2 The required strength of capacity-protected elements (columns, beams, diagonal braces and connections) shall be determined based on the expected over-strength capacity of the link to be taken as $1.1R_yS_h$ times the design strength of the link as per **11.2**, where S_h (a factor to account for strain hardening and strain rate) is equal to 1.25 for I-shaped links and 1.4 for box shaped links.

Exception — When the link and beam have the same section and is continuous:

a) the required strength of the beam outside the link shall be determined based on 0.9 times the expected link overstrength capacity; and
b) the design capacity of the beam shall be calculated based on the expected material yield stress.

12.3.3 *System Requirements*

The requirements specified hereunder shall be satisfied.

12.3.3.1 *Link rotation angle*

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift. The link rotation angle shall not exceed 0.08 rad.

12.3.3.2 *Bracing of link*

Bracing shall be provided at both the top and bottom flanges of the link at the ends of the link for I-shaped sections. Such bracings shall have stiffness and strength as specified in **6.3.3**.

12.3.4 *Member Requirements*

The requirements specified hereunder shall be satisfied.

12.3.4.1 *Basic requirements*

Brace members, beams outside the links and columns shall satisfy width-to-thickness limitations specified in **5.3**. Apart from columns, the beams and braces in EBFs may be subjected to significant axial and bending forces; hence their design capacities shall be determined as for beam-column members as per IS 800.

12.3.4.2 *Links*

The link length *e* shall be considered to extend from brace connection to brace connection for centre links. In addition, requirements specified in **11** shall apply.

12.3.4.3 *Protected zones*

The protected zones of EBFs are the links. Use of shear studs on the links is prohibited.

12.3.4.4 *Beam-to-column connections*

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection assembly shall be designed to resist beam moment taken equal to $1.1R_y f_{yb} Z_{pb}$. Also, the sum of the expected column flexural strengths shall exceed $1.1R_y f_{yb} Z_{pb}$.

This connection moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength earthquake load.

12.3.4.5 *Braces*

Braces in EBFs shall be designed not to yield in tension or buckle in compression corresponding to shear force in link taken equal to $1.2R_y$ times the design strength of connected link as per **11.2**, and shall satisfy the requirements of **5.4**.

12.3.4.6 *Brace connections*

When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the earthquake load effect determined using the overstrength earthquake load specified in **5.5**. Connections of braces designed to resist a portion of the link end moment shall be designed as fully restrained.

12.3.4.7 *Column splices*

Requirements specified in **7.5** shall apply.

In addition, column splices shall be designed to develop at least 50 percent of the plastic flexural strength, M_p, of the smaller connected member, and have shear strength greater than $\sum M_p / H_c$.

where

$\sum M_p$ = Sum of the plastic flexural strengths, at the top and bottom ends of the column; and

H_c = Clear height of the column between beam connections.

12.3.4.8 *Demand critical welds*

The following welds shall be designed as demand critical welds:

a) Groove welds at column splices;
b) Welds at column-to-base plate connections, except when:
    1) Column plastic hinge at, or near the base plate is precluded by conditions of restraint, or
    2) There is no net tension under load combinations including the overstrength earthquake load;
c) Welds in beam-column connections;
d) Connections of braces designed to resist a portion of the link end moment; and
e) In built-up beams, welds within the link connecting the webs to the flanges.

ANNEX A

(*Clause* 2)

LIST OF REFERRED STANDARDS

IS No.	*Title*
IS 800 : 2007	General construction in steel — Code of practice (*third revision*)
IS 808 : 2021	Hot rolled steel beam column channel and angle sections — Dimensions and properties
IS 875	Code of practice for design loads (other than earthquake) for buildings and structures:
(Part 1) : 1987	Dead loads — Unit weight of building material and stored materials (*second revision*)
(Part 2) : 1987	Imposed loads — Code of practice (*second revision*)
IS 1363	Hexagon head bolts, screws and nuts of product grade 'C':
(Part 1) : 2019	Hexagon head bolts (size range M5 to M64) (*fifth revision*)
(Part 2) : 2018	Hexagon head screws (size range M5 to M64) (*fifth revision*)
(Part 3) : 2018	(Style 1) hexagon nuts (size range M5 to M64) (*fifth revision*)
IS 1364	Hexagon head bolts, screws and nuts of product grades A and B:
(Part 1) : 2018	Hexagon head bolts (size range M1.6 to M64) (*fifth revision*)
(Part 2) : 2018	Hexagon head screws (size range M1.6 to M64) (*fifth revision*)
(Part 3) : 2018	Hexagon nuts (style 1) (size range M1.6 to M64)
(Part 4) : 2003	Hexagon thin nuts (chamfered) (size range M1.6 to M64) (*fourth revision*)
(Part 5) : 2002	Hexagon thin nuts — Product grade B (unchamfered) (size range M1.6 to M10) (*fourth revision*)
(Part 6) : 2018	Hexagon nuts (style 2) (*first revision*)
IS 1893 (Part 1) : 2016	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*sixth revision*)
IS 2062 : 2011	Hot rolled medium and high tensile structural steel — Specification (*seventh revision*)
IS 3640 : 1982	Specification for hexagon fit bolts (*first revision*)
IS 3757 : 1985	Specification for high strength structural bolts (*second revision*)
IS 4000 : 1992	High strength bolts in steel structures — Code of practice (*first revision*)
IS 5624 : 2021	Foundation bolts specification (*second revision*)
IS 6610 : 1972	Specification for heavy washers for steel structures
IS 6639 : 1972	Specification for hexagon bolts for steel structures
IS 6649 : 1985	Specification for hardened and tempered washers for high strength structural bolts and nuts (*first revision*)

ANNEX B

(*Foreword*)

COMMITTEE COMPOSITION

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