Comments on Earthquake Resistant Design and Detailing of Steel Buildings - Code of Practice

.	Clause / Subclause / Para No.	Comments / Suggestions	Modified Wording	Reasons / Justification
	1 SCOPE 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 (SM~~R~~F); b) Special concentrically braced frame (SCBF); and c) Eccentrically braced frame (EBF). In seismic zones IV and V, ~~all steel buildings shall be EBF systems~~SCBF shall not be used.	The provisions of this standard apply for design and detailing of steel buildings having the following structural systems: a) Special moment frame (SMF); b) Special concentrically braced frame (SCBF); and c) Eccentrically braced frame (EBF). In seismic zones IV and V, SCBF shall not be used.	Replace SMRF with SM for consistency with IS 800 Present wording implies that SMF cannot be used in zones IV and V. SMRF is not prohibited in zones IV and V (See clause 12.1.1)
	5.2 Materials 5.2 (b)	Any other equivalent grades of steel used in seismic force resisting systems shall satisfy the following: 1) Characteristic yield stress fy of structural steel sections and plates in which inelastic action is expected shall not exceed 350 MPa;	Any other equivalent grades of steel used in seismic force resisting systems shall satisfy the following: 1) Characteristic yield stress fy of structural steel sections and plates in which inelastic action is expected shall not exceed 350 MPa;	Gravity framing systems need not conform to this requirement.

		2) Structural steel shall have a minimum elongation of 22 percent. 3) Structural steel shall have a minimum Charpy V-Notch impact test value of 27 J at 0 °C. 4) The ratio of the ultimate strength to the yield strength shall at least be 1.15.	2) Structural steel shall have a minimum elongation of 22 percent. 3) Structural steel shall have a minimum Charpy V-Notch impact test value of 27 J at 0 °C. 4) The ratio of the ultimate strength to the yield strength shall at least be 1.15.	
	5.2 Materials 5.2 (c)	Table 1 Material Strength Uncertainty Factors Ry and Ru *(Needs revision)*	Similar to AISC 341-22 Table A3.2 including Ry and Ru for concrete and reinforcement	Ry and Ru depend on material as also on the manufacturing process so should be listed based on the end product such as hot rolled sections, HSS, plates and strips.
	5.2 Materials 5.2.2	All welds of seismic force resisting systems, 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 27 J at -20 °C.	All welds of seismic force resisting systems, 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 27 J at -20 °C	Gravity framing systems need not conform to this requirement.
	5.3 Section Classification	Table 2 Limiting Width-to-Thickness Ratios for Compression Elements for Seismic Applications *(Needs revision)*	Similar to AISC 341-22 Table D1.1b	Consistent with the *proposed* revision of AISC 341-22

	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 columns, beams in SCBFs and EBFs, braces in EBFs, and all connections which are part of lateral load resisting system in steel buildings: $1.2DL + \gamma_{LL}LL \pm 1.0EL_m$ and $0.9DL \pm 1.0EL_m$ Where EL_m = Estimated maximum equivalent earthquake force induced in the structure = 'Ω x EL but not greater than capacity-limited force (See Appendix C) DL = Dead load as per IS 875 (Part 1) LL = Live load as per IS 875 (Part 2) EL = Earthquake load as per IS 1893 'Ω = Overstrength factor = 2.5 for SCBFs and EBFs; and	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 columns, beams in SCBFs and EBFs, braces in EBFs, and all connections which are part of lateral load resisting system in steel buildings: $1.2DL + \gamma_{LL}LL \pm 1.0EL_m$ and $0.9DL \pm 1.0EL_m$ Where EL_m = Estimated maximum equivalent earthquake force induced in the structure = 'Ω x EL but not greater than capacity-limited force (See Appendix C) DL = Dead load as per IS 875 (Part 1) LL = Live load as per IS 875 (Part 2) EL = Earthquake load as per IS 1893 'Ω = Overstrength factor = 2.5 for SCBFs and EBFs; and	There are several triggers and exceptions on the applicability of overstrength factor mentioned in ASCE 7 but not included in IS 1893. Capacity-limited seismic force is an alternative and more accurate method of estimating maximum equivalent earthquake force. One whole appendix needs to be dedicated to the applicability of overstrength factor and calculation of capacity-limited force.

		3.0 for SMRFs; γ_{LL} = Partial safety factor for live load = 0.25 for LL class ≤ 3 kN/m^2 = 0.50 for LL class > 3 kN/m^2	3.0 for SMRFs; γ_{LL} = Partial safety factor for live load = 0.25 for LL class ≤ 3 kN/m^2 = 0.50 for LL class > 3 kN/m^2	
.	6 BEAMS 6.2 Slenderness	The ratio of the maximum unbraced length of the compression flange of a beam Lbr to the radius of gyration ry about the weaker axis of the beam cross-section shall not exceed 25 for a distance of 2db from plastic hinge ~~for buildings in Seismic Zones III, IV and V~~. Lbr / ry for the remaining portion of the beam shall not exceed 0.19E/(Ry.fyb)	The ratio of the maximum unbraced length of the compression flange of a beam Lbr to the radius of gyration ry about the weaker axis of the beam cross-section shall not exceed 25 for a distance of 2db from plastic hinge. Lbr / ry for the remaining portion of the beam shall not exceed 0.19E/(Ry.fyb)	“exceed 25 for a distance of 2db” does not specify the location to which it applies. “For buildings in zones ..” is redundant as the code applies to these zones only. It may be noted that the Lbr/ry ratio is limited to 0.086E/(RyFy) in case of highly ductile members in AISC 341-22 draft. Also the draft has specific requirements for strength and stiffness of bracing adjacent to plastic hinge
	6 BEAMS 6.3 Bracing	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~~ relative panel or point torsional bracing.	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 relative panel or point torsional bracing.	Allow alternative bracing method as relative panel bracing will be very costly.

	6 BEAMS 6.4 Strength	The design strength of beam shall satisfy the load combinations in IS 1893 (Part 1), except the overstrength load combinations specified in 5.5. Exception: Where a beam is a part of a diaphragm collector or chord, overstrength load combinations specified in 5.5 shall apply.	The design strength of beam shall satisfy the load combinations in IS 1893 (Part 1), except the overstrength load combinations specified in 5.5. Exception: Where a beam is a part of a diaphragm collector or chord, overstrength load combinations specified in 5.5 shall apply.	Diaphragm collectors and chords are expected to remain elastic.
	7 COLUMNS ~~7.4.2~~	~~Columns in buildings designed to resist lateral loads shall not carry tensile forces.~~		Basis for this clause is not clear.
	7 COLUMNS 7.5 Splice	Column splices shall be located in the middle third of the storey, at least 1.0 m away from the beam-to-column connection. Exception: When the column clear height between beam-to-column connections is less than 3.0 m, splices shall be at half the clear height.	Column splices shall be located in the middle third of the storey, at least 1.0 m away from the beam-to-column connection. Exception: When the column clear height between beam-to-column connections is less than 3.0 m, splices shall be at half the clear height.	Present clause cannot be applied to columns shorter than 3.0 m

	7 COLUMNS 7.5.1	~~Strength of Splice~~ ~~The design strength of column splices shall at least be 1.80 times the required strength. Further, design strength of each flange splice plate shall at least be 1.2 Ry fy Af where Af is the area of the smaller flange (when applicable).~~		This clause overlaps and contradicts clauses 12.1.4.6, 12.2.4.6 and 12.3.4.7
	8 BEAM COLUMN JOINT 8.2 Column to Beam Strength Ratio	At a beam–column joint, the following strength ratio shall be satisfied: $\frac{\Sigma Mpc}{\Sigma Mbo} = \frac{\Sigma Zpc\, fyc\, (1-\frac{Pu}{Pd})}{\Sigma 1.1 Ry\, Zpb\, fyb} \geq 1.8$ *Delete above equation and replace by* $\frac{\Sigma Mpc}{\Sigma Mbo} = \frac{\Sigma Zpc\,(fyc - Pr/Ag)}{\Sigma(1.1 Ry Zpb fyb + Mv)} > 1.0$ where Zpc and Zpb are the plastic section modul~~us~~ii and fyc and fyb are the characteristic yield strength of column and beam cross-sections respectively, ~~Pu is the maximum factored axial compressive load and Pd is the design strength under axial compression, and~~ Pr is the compressive axial strength as	At a beam–column joint, the following strength ratio shall be satisfied: $\frac{\Sigma Mpc}{\Sigma Mbo} = \frac{\Sigma Zpc\,(fyc - Pr/Ag)}{\Sigma(1.1 Ry Zpb fyb + Mv)} > 1.0$ where Zpc and Zpb are the plastic section modulii and fyc and fyb are the characteristic yield strength of column and beam cross-sections respectively, Pr is the compressive axial strength as determined using the overstrength seismic load neglecting applied moments unless the moment results from a load applied to the column between points of lateral support, Ag is the gross area of the column,	Revised numerator is more explicit. Denominator must include a term Mv to account for additional moment due to shear amplification from the centre of the plastic hinge to the centerline of the column.

		determined using the overstrength seismic load neglecting applied moments unless the moment results from a load applied to the column between points of lateral support, Ag is the gross area of the column, Ry is the material uncertainty factor corresponding to the grade of steel in beams and Mv is the additional moment due to shear amplification from the centre of the plastic hinge to the centerline of the column. .	Ry is the material uncertainty factor corresponding to the grade of steel in beams and Mv is the additional moment due to shear amplification from the centre of the plastic hinge to the centerline of the column.	
	8 BEAM COLUMN JOINT 8.2 Column to Beam Strength Ratio 8.2.1	The above requirement need not apply if ~~be~~ the following conditions in (a) or (b) are satisfied (a) Columns with $Prc < 0.3fycAg$ for all load combinations other than those determined using the overstrength seismic load and that satisfy either of the following: (1) Columns at the roof level (2) Columns where (i) the sum of the available shear strengths of all exempted columns in the story is less than	The above requirement need not apply if the following conditions in (a) or (b) are satisfied (a) Columns with $Prc < 0.3fycAg$ for all load combinations other than those determined using the overstrength seismic load and that satisfy either of the following: (1) Columns at the roof level (2) Columns where (i) the sum of the available shear strengths of all exempted columns in the story is less than	Three exceptions are given. In the first exception, columns with low axial loads used in one-story buildings or in the top story of a multi-story building need not satisfy Cl. 8.2 because concerns for inelastic soft or weak stories are not significant in such cases. Additionally, exception is made for columns with low axial loads, under certain conditions, in order to provide design flexibility where the requirement of Cl.8.2 would be impractical, such as at large transfer girders. Finally, Cl. 8.2.1(b) provides an exception for columns in levels that are significantly stronger than in the level above because

		20% of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction, and (ii) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33% of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10% of the plan dimension perpendicular to the line of columns. Prc is the required axial strength for all combinations other than that required by Cl. 5.5 above. (b) Columns in any story that has a	20% of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction, and (ii) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33% of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10% of the plan dimension perpendicular to the line of columns. Prc is the required axial strength for all combinations other than that required by Cl. 5.5 above. (b) Columns in any story that has a	column yielding at the stronger level would be unlikely.

		ratio of available shear strength to required shear strength that is 50% greater than the story above. For purposes of this clause, the available shear strengths of the columns should be calculated as the limit strengths considering the flexural strength at each end as limited by the flexural strength of the attached beams, or the flexural strength of the columns themselves, divided by H, where H is the story height.	ratio of available shear strength to required shear strength that is 50% greater than the story above. For purposes of this clause, the available shear strengths of the columns should be calculated as the limit strengths considering the flexural strength at each end as limited by the flexural strength of the attached beams, or the flexural strength of the columns themselves, divided by H, where H is the story height.	
	8 BEAM COLUMN JOINT 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 Ry fy Zpb, and shear demand determined based on capacity design principle considering, (i) beams to be bending in double curvature, (ii) plastic hinges of strength 1.1 Ry fy Zpb assumed to act at a distance db/2 from the end of the connection, and (iii) gravity load required to be carried. All beam-column connections shall conform to the requirements of	Fully-restrained beam–column connections shall be used in moment frames, capable of transferring at least a bending moment of 1.1 Ry fy Zpb, and shear demand determined based on capacity design principle considering, (i) beams to be bending in double curvature, (ii) plastic hinges of strength 1.1 Ry fy Zpb assumed to act at a distance db/2 from the end of the connection, and (iii) gravity load required to be carried. All beam-column connections shall conform to the requirements of	Reduced Beam Section (RBS) is a common moment frame connection whereby, instead of reinforcing the connection, beam is weakened. Prequalified connections for seismic zones need to be described in an appendix similar to AISC 358.

		Appendix A.	Appendix A.	
	8.4.1 Welded Beam Column Connection	*Delete all descriptions and figures and add the following sentence:* All welding shall conform to Appendix B.	All welding shall conform to Appendix B.	Welding for seismic zones needs to be described in an appendix similar to AWS D1.8
	8.4.2 Bolted Beam Column Connection	*New sub-clause to be added*		Allow alternative bolted connection as site welding is costly and requires expertise.
	12.1 Special Moment Resisting Frames 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 [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.	Special moment 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 frames may be used in any seismic zones [see IS 1893 (Part 1)] and for any buildings (importance factor values) Yielding of beam to column connections in SMFs shall not be permitted by this standard.	Yielding at column base is acceptable by all codes including this (See clause 12.1.4.7) as it forms a global mechanism.
	12.1 Special	Beams in SMRFs are permitted to	Beams in SMFs are permitted to carry	Properly designed composite

	Moment Resisting Frames 12.1.3.1 Beams	carry gravity and lateral loads through composite action with reinforced concrete slab. ~~For lateral load action, composite action shall not be considered. Also, 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.~~	gravity and lateral loads through composite action with reinforced concrete slab.	diaphragms are critical components of seismic force resisting systems
	12.1 Special Moment Resisting Frames 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. ~~Further, steel headed stud anchors shall not be placed on beam flanges within the protected zone~~ Discontinuities resulting from fabrication and erection procedures and from other attachments are prohibited in the region. Exception: Welded steel headed stud anchors and other connections are permitted in protected zones when designed as a prequalified connection (See appendix A)	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. Discontinuities resulting from fabrication and erection procedures and from other attachments are prohibited in the region. Exception: Welded steel headed stud anchors and other connections are permitted in protected zones when designed as a prequalified connection (See appendix A)	Discontinuities in protected zones need to be avoided but blanket ban on all forms of discontinuities is counterproductive. Composite diaphragms depend on steel headed anchors and other connectors for load transfer.

	12.1 Special Moment ~~Resisting~~ Frames 12.1.4.6 Strength of Column Splices	~~Column Splices shall be located in middle third of the storey, at least 1.0 m away from the beam-to-column connection. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds.~~ The column splices shall be designed at least for developing the flexural strength of the smaller column cross section at the splice joint, considering Ry, and associated shear force acting simultaneously. ~~When welds are used for flange and web splices, they shall be complete joint penetration groove weld type.~~	The column splices shall be designed at least for developing the flexural strength of the smaller column cross section at the splice joint, considering Ry , and associated shear force acting simultaneously.	Splice location and weld specs are defined once in clause 7.5 Rationale for only CJP welds is not clear.
	12.2 Special Concentrically Braced Frames 12.2.3.3 Continuity of load path	For the purpose of this standard, a line of ~~braces~~ seismic force resisting frames 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~~ seismic force resisting frames interconnected adequately through rigid diaphragm ~~-A diaphragm shall be considered to be rigid if the maximum~~	For the purpose of this standard, a line of seismic force resisting frames 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 seismic force resisting frames interconnected adequately through rigid diaphragm as defined in IS1893. When such plan offset exceeds 10 percent, diaphragms must	This clause should preferably be a part of a separate section or appendix on diaphragm design and apply to all seismic force resisting frames, not just concentrically braced frames. Definition of rigid diaphragm overlaps IS1893 provision so should be avoided.

		~~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~~ as defined in IS1893. When such plan offset exceeds 10 percent, diaphragms must be designed to transfer seismic forces caused by horizontal offsets	be designed to transfer seismic forces caused by horizontal offsets	Ten percent offset is the limit up to which additional diaphragm forces need not be considered. In certain cases, plan offset may be greater than 10 percent.
	12.2 Special Concentrically Braced Frames 12.2.4.6 Strength of Column Splices	~~Column splices shall be located in middle third of the storey, at least 1.0 m away from the beam-to-column connection. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds.~~ Column splices shall be designed to develop at least 50 percent of the plastic flexural strength, Mp, of the connected members, and have shear strength greater than $\Sigma Mp / Hc$ where Hc = clear height of the column between beam connections	Column splices shall be designed to develop at least 50 percent of the plastic flexural strength, Mp, of the connected members, and have shear strength greater than $\Sigma Mp / Hc$ where Hc = clear height of the column between beam connections ΣMp = sum of the plastic flexural strengths, at the top and bottom ends of the column	Splice location and weld specs are defined once in clause 7.5

		ΣMp = sum of the plastic flexural strengths, at the top and bottom ends of the column		
	12.3 Eccentrically Braced Frame 12.3.4.7 Strength of Column Splices	~~Column splices shall be located in middle third of the storey, at least 1.0 m away from the beam-to-column connection. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds.~~ Column splices shall be designed to develop at least 50 percent of the plastic flexural strength, Mp, of the connected members, and have shear strength greater than $\Sigma Mp/Hc$ where Hc = clear height of the column between beam connections ΣMp = sum of the plastic flexural strengths, at the top and bottom ends of the column	Column splices shall be designed to develop at least 50 percent of the plastic flexural strength, Mp, of the connected members, and have shear strength greater than $\Sigma Mp/Hc$ where Hc = clear height of the column between beam connections ΣMp = sum of the plastic flexural strengths, at the top and bottom ends of the column	Splice location and weld specs are defined once in clause 7.5

Appendix A Prequalified Connections

(Refer AISC 358)

Appendix B Structural Welding

(Refer AWS D1.8)

Appendix C Overstrength Factor ’Ω and Capacity-Limited Force

(Refer ASCE 7)

Appendix D Diaphragms, Chords and Collectors

(Refer ASCE 7)