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Seismic Provisions for Structural Steel Buildings

June 15, 1992

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R Res[illegible]e modification factor. (3)

R' Loa[illegible]e to initial rainwater or ice exclusive of the ponding contribution, kips. (Symbol R is used in the *Specification*). (3)

R_n Nominal strength of a member. (8)

S Snow load, kips. (3)

V Base shear due to earthquake load, kips. (3)

V_n Nominal shear strength of a member, kips. (8)

V_u Required shear strength on a member, kips. (8)

V_p Nominal shear strength of an active link, kips. (10)

V_{pa} Nominal shear strength of an active link modified by the axial load magnitude, kips. (10)

W Wind load, kips. (3)

W_g Total weight of the building, kips. (3)

Z_b Plastic section modulus of a beam, in.^{3} (8)

Z_c Plastic section modulus of a column, in.^{3} (8)

b Width of compression element, in. (Table 8-1)

b_f Flange width, in. (8)

b_{cf} Column flange width, in. (8)

d_b Overall beam depth, in. (8)

d_c Overall column depth, in. (8)

d_z Overall panel zone depth between continuity plates, in. (8)

e EBF link length, in. (10)

h Assumed web depth for stability, in. (Table 8-1)

r Governing radius of gyration, in. (9)

r_y Radius of gyration about y axis, in. (8)

t_{bf} Thickness of beam flange, in. (8)

t_{cf} Thickness of column flange, in. (8)

t_f Thickness of flange, in. (8)

t_p Thickness of panel zone including doubler plates, in. (8)

t_w Thickness of web, in. (8)

t_z Thickness of panel zone (doubler plates not necessarily included), in. (8)

w_z Width of panel zone between column flanges, in. (8)

α Fraction of member force transferred across a particular net section. (9)

ρ Ratio of required axial force P_u to required shear strength V_u of a link. (10)

k Slenderness parameter. (9)

k_p Limiting slenderness parameter for compact element. (8)

k_r Limiting slenderness parameter for non-compact element. (9)

ϕ Resistance factor. (6,10)

ϕ_b Resistance factor for beams. (6)

ϕ_c Resistance factor for columns in compression. (6,10)

ϕ_t Resistance factor for columns in tension. (6)

ϕ_v Resistance factor for shear strength of panel zone of beam-to-column connections. (8)

ϕ_w Resistance factor for welds. (6)

Glossary

Beam. A structural member whose primary function is to carry loads transverse to it[s] longitudinal axis, usually a horizontal member in a seismic frame system.

Braced Frame. An essentially vertical truss system of concentric or eccentric type tha[t] resists lateral forces on the structural system.

Concentrically Braced Frame (CBF). A braced frame in which all members of th[e] bracing system are subjected primarily to axial forces. The CBF shall meet th[e] requirements of Sect. 9.

Connection. Combination of joints used to transmit forces between two or mor[e] members. Categorized by the type and amount of force transferred (moment shear, end reaction).

Continuity Plates. Column stiffeners at top and bottom of the panel zone.

Design strength. Resistance (force, moment, stress, as appropriate) provided b[y] element or connection; the product of the nominal strength and the resistanc[e] factor.

Diagonal Bracing. Inclined structural members carrying primarily axial load employed to enable a structural frame to act as a truss to resist horizontal loads.

Dual System. A dual system is a structural system with the following features:

- An essentially complete space frame which provides support for gravity loads
- Resistance to lateral load is provided by moment resisting frames (SMF) o[r] (OMF) which is capable of resisting at least 25 percent of the base shear an[d] concrete or steel shear walls, steel eccentrically (EBF) or concentrically (CBF) braced frames.
- Each system shall be also designed to resist the total lateral load in proportion to its relative rigidity.

Eccentrically Braced Frame (EBF). A diagonal braced frame in which at least one end of each bracing member connects to a beam a short distance from a beam-to-column connection or from another beam-to-brace connection. The EBF shall mee[t] the requirements of Sect. 10.

Essential Facilities. Those facilities defined as essential in the applicable code under which the structure is designed. In the absence of such a code, see ASCE 7-92.

Joint. Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer.

K Braced Frame. A concentric braced frame (CBF) in which a pair of diagonal braces located on one side of a column is connected to a single point within the clear column height.

Lateral Support Member. Member designed to inhibit lateral buckling or lateral-torsional buckling of primary frame members.

Link. In EBF, the segment of a beam which extends from column to column, located between the end of a diagonal brace and a column or between the ends of two diagonal braces of the EBF. The length of the link is defined as the clear distance between the diagonal brace and the column face or between the ends of two diagonal braces.

Link Intermediate Web Stiffeners. Vertical web stiffeners placed within the link.

Link Rotation Angle. The link rotation angle is the plastic angle between the link and the beam outside of the link when the total story drift is E' / E times the drift derived using the specified base shear, V.

Link Shear Design Strength. The lesser of ϕV_p or $2\phi M_p/e$, where $\phi = 0.9$, $V_p = 0.55F_y dt_w$ and e = the link length except as modified by Sect. S9.2.f.

LRFD. (Load and Resistance Factor Design). A method of proportioning structural components (members, connectors, connecting elements, and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all design load combinations.

Moment Frame. A building frame system in which seismic shear forces are resisted by shear and flexure in members and joints of the frame.

Nominal loads. The magnitudes of the loads specified by the applicable code.

Nominal strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects, and differences between laboratory and field conditions.

Ordinary Moment Frame (OMF). A moment frame system which meets the requirements of Sect. 7.

P - Delta effect. Secondary effect of column axial loads and lateral deflection on the shears and moments in members.

Panel Zone. Area of beam-to-column connection delineated by beam and column flanges.

Required Strength. Load effect (force, moment, stress, as appropriate) acting on element of connection determined by structural analysis from the factored loads (using most appropriate critical load combinations).

Resistance Factor. A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

Slip-Critical Joint. A bolted joint in which slip resistance of the connection is required.

Special Moment Frame (SMF). A moment frame system which meets the requirements of Sect. 8.

Structural System. An assemblage of load-carrying components which are joined together to provide regular interaction or interdependence.

V Braced Frame. A concentrically braced frame (CBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an Inverted V Braced Frame.

X Braced Frame. A concentrically braced frame (CBF) in which a pair of diagonal braces crosses near mid-length of the braces.

Y Braced Frame. An eccentrically braced frame (EBF) in which the stem of the Y is the link of the EBF system.

Seismic Provisions for Structural Steel Buildings

June 15, 1992

Part I—Load and Resistance Factor Design (LRFD)

1. SCOPE

These special seismic requirements are to be applied in conjunction with the AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (LRFD), 1986; hereinafter referred to as the *Specification.* They are intended for the design and construction of structural steel members and connections in buildings for which the design forces resulting from earthquake motions have been determined on the basis of energy dissipation in the non-linear range of response.

Seismic provisions and the nominal loads for each Seismic Performance Category, Seismic Hazard Exposure Group, or Seismic Zone shall be as specified by the applicable code under which the structure is designed or where no code applies, as dictated by the conditions involved. In the absence of a code, the Performance Categories, Seismic Hazard Exposure Groups, loads and load combinations shall be as given herein.

2. SEISMIC PERFORMANCE CATEGORIES

Seismic Performance Categories vary with the Seismic Hazard Exposure Group shown in Table 2-1, the Effective Peak Velocity Related Acceleration, A_v, and the Seismic Hazard Exposure Group shown in Table 2-2.

In addition to the general requirements assigned to the various Seismic Performance Categories in the applicable building code for all types of construction, the following requirements apply to fabricated steel construction for buildings and structures with similar structural characteristics.

2.1. Seismic Performance Categories A, B, and C

Buildings assigned to Categories A, B, and C, except Category C in Seismic Hazard Exposure Group III where the value of $A_v \geq 0.10$, shall be designed either in accordance with solely the *Specification* or in accordance with the *Specification* and these provisions.

TABLE 2-1
Seismic Hazard Exposure Groups

Group III	Buildings having essential facilities that are necessary for post-earthquake recovery and requiring special requirements for access and functionality.
Group II	Buildings that constitute a substantial public hazard because of occupancy or use.
Group I	All buildings not classified in Groups II and III.

2.2. Seismic Performance Category C

Buildings assigned to Category C in Seismic Hazard Exposure Group III where the value of $A_v \geq 0.10$ shall be designed in accordance with the *Specification* as modified by the additional provisions of this section.

2.2.a. Steel used in seismic resisting systems shall be limited by the provisions of Sect. 5.

2.2.b. Columns in seismic resisting systems shall be designed in accordance with Sect. 6.

2.2.c. Ordinary Moment Frames (OMF) shall be designed in accordance with the provisions of Sect. 7.

2.2.d. Special Moment Frames (SMF) are required to conform only to the requirements of Sects. 8.2, 8.7, and 8.8.

2.2.e. Braced framed systems shall conform to the requirements of Sects. 9 or 10 when used alone or in combination with the moment frames of the seismic resisting system.

2.2.f. A quality assurance plan shall be submitted to the regulatory agency for the seismic force resisting system of the building.

2.3. Seismic Performance Categories D and E

Buildings assigned to Categories D and E shall be designed in accordance with the *Specification* as modified by the additional provisions of this section.

2.3.a. Steel used in seismic resisting systems shall be limited by the provisions of Sect. 5.

2.3.b. Columns in seismic resisting systems shall be designed in accordance with Sect. 6.

2.3.c. Ordinary Moment Frames (OMF) shall be designed in accordance with the provisions of Sect. 7.

2.3.d. Special Moment Frames (SMF) shall be designed in accordance with the provisions of Sect. 8.

2.3.e. Braced framed systems shall conform to the requirements of Sects. 9. (CBF) or 10. (EBF) when used alone or in combination with the moment frames of the seismic resisting system.

The use of K-bracing systems shall not be permitted as part of the seismic resisting system except as permitted by Sect. 9.5. (Low Buildings)

2.3.f. A quality assurance plan shall be submitted to the regulatory agency for the seismic force resisting system of the building.

TABLE 2-2
Seismic Performance Categories

Value of A_v	Seismic Hazard Exposure Group I	II	III
$0.20 \leq A_v$	D	D	E
$0.15 \leq A_v < 0.20$	C	D	D
$0.10 \leq A_v < 0.15$	C	C	C
$0.05 \leq A_v < 0.10$	B	B	C
$A_v < 0.05$	A	A	A

3. LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS

3.1. Loads and Load Combinations

The following specified loads and their effects on the structure shall be taken into account:

D : dead load due to the weight of the structural elements and the permanent features on the structure.
L : live load due to occupancy and moveable equipment.
L_r : roof live load.
W : wind load.
S : snow load.
E : earthquake load (where the horizontal component is derived from base shear Formula $V = C_s W_g$).
R' : load due to initial rainwater or ice exclusive of the ponding contribution.

In the Formula $V = C_s W_g$ for base shear:

C_s = Seismic design coefficient
W_g = Total weight of the building, see the applicable code.

For the nominal loads as defined above, see the applicable code.

The required strength of the structure and its elements shall be determined from the appropriate critical combination of factored loads. The following Load Combinations and corresponding load factors shall be investigated:

$$1.4D \quad (3\text{-}1)$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R') \quad (3\text{-}2)$$

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R') + (0.5L \text{ or } 0.8W) \quad (3\text{-}3)$$

$$1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R') \quad (3\text{-}4)$$

$$1.2D \pm 1.0E + 0.5L + 0.2S \quad (3\text{-}5)$$

$$0.9D \pm (1.0E \text{ or } 1.3W) \quad (3\text{-}6)$$

Exception: The load factor on L in Load Combinations 3-3, 3-4, and 3-5 shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

Other special load combinations are included with specific design requirements throughout these provisions.

Orthogonal earthquake effects shall be included in the analysis unless noted specifically otherwise in the governing building code.

Where required by these provisions, an amplified horizontal earthquake load of $0.4R \times E$ (where the term $0.4R$ is greater or equal to 1.0) shall be applied in lieu of the horizontal component of earthquake load E in the load combinations above. The term R is the earthquake response modification coefficient contained in the applicable code. The additional load combinations using the amplified horizontal earthquake load are:

$$1.2D + 0.5L + 0.2S \pm 0.4R \times E \quad (3\text{-}7)$$

$$0.9D \pm 0.4R \times E \quad (3\text{-}8)$$

Exception: The load factor on L in Load Combinations 3-7 shall equal 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.

The term $0.4R$ in Load Combinations 3-7 and 3-8 shall be greater or equal to 1.0.

Where the amplified load is required, orthogonal effects are not required to be included.

3.2. Nominal Strengths

The nominal strengths shall be as provided in the *Specification.*

4. STORY DRIFT

Story drift shall be calculated using the appropriate load effects consistent with the structural system and the method of analysis. Limits on story drift shall be in accordance with the governing code and shall not impair the stability of the structure.

5. MATERIAL SPECIFICATIONS

Steel used in seismic force resisting systems shall be as listed in Sect. A3.1 of the *Specification*, except for buildings over one story in height. The steel used in seismic resisting systems described in Sections 8, 9, and 10 shall be limited to the following ASTM Specifications: A36, A500 (Grades B and C), A501, A572 (Grades 42 and 50), and A588. The steel used for base plates shall meet one of the preceding ASTM Specifications or ASTM A283 Grade D.

6. COLUMN REQUIREMENTS

6.1. Column Strength

When $P_u / \phi P_n > 0.5$, columns in seismic resisting frames, in addition to complying with the *Specification*, shall be limited by the following requirements:

6.1.a. Axial compression loads:

$$1.2P_D + 0.5P_L + 0.2P_S + 0.4R \times P_E \leq \phi_c P_n \quad (6\text{-}1)$$

where the term $0.4R$ is greater or equal to 1.0.

Exception: The load factor on P_L in Load Combination 6-1 shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

6.1.b. Axial tension loads:

$$0.9P_D - 0.4R \times P_E \leq \phi_t P_n \quad (6\text{-}2)$$

where the term $0.4R$ is greater or equal to 1.0.

6.1.c. The axial Load Combinations 6-1 and 6-2 are not required to exceed either of the following:

1. The maximum loads transferred to the column, considering 1.25 times the design strengths of the connecting beam or brace elements of the structure.
2. The limit as determined by the foundation capacity to resist overturning uplift.

6.2. Column Splices

Column splices shall have a design strength to develop the column axial loads given in Sect. 6.1.a, b, and c as well as the Load Combinations 3-1 to 3-6.

6.2.a. In column splices using either complete or partial penetration welded joints, beveled transitions are not required when changes in thickness and width of flanges and webs occur.

6.2.b. Splices using partial penetration welded joints shall not be within 3 ft of the beam-to-column connection. Column splices that are subject to net tension forces shall comply with the more critical of the following:

1. The design strength of partial penetration welded joints, the lesser of $\phi_w F_w A_w$ or $\phi_w F_{BM} A_w$, shall be at least 150 percent of the required strength, where $\phi_w = 0.8$ and $F_w = 0.6F_{EXX}$.
2. The design strength of welds shall not be less than $0.5F_{yc}A_f$, where F_{yc} is the yield strength of the column material and A_f is the flange area of the smaller column connected.

7. REQUIREMENTS FOR ORDINARY MOMENT FRAMES (OMF)

7.1. Scope

Ordinary Moment Frames (OMF) shall have a design strength as provided in the *Specification* to resist the Load Combinations 3-1 through 3-6 as modified by the following added provisions:

7.2. Joint Requirements

All beam-to-column and column to beam connections in OMF which resist seismic forces shall meet one of the following requirements:

7.2.a. FR (fully restrained) connections conforming with Sect. 8.2, except that the required flexural strength, M_u, of a column-to-beam joint is not required to exceed the nominal plastic flexural strength of the connection.

7.2.b. FR connections with design strengths of the connections meeting the requirements of Sect. 7.1 using the Load Combinations 3-7 and 3-8.

7.2.c. Either FR or PR (partially restrained) connections shall meet the following:

1. The design strengths of the members and connections meet the requirements of Sect. 7.1.
2. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at a story drift calculated at a horizontal load of $0.4R \times E$, (where the term $0.4R$ is equal to or greater than 1.0).
3. The additional drift due to PR connections shall be considered in design.

FR and PR connections are described in detail in Sect. A2 of the *Specification.*

8. REQUIREMENTS FOR SPECIAL MOMENT FRAMES (SMF)

8.1. Scope

Special Moment Frames (SMF) shall have a design strength as provided in the *Specification* to resist the Load Combinations 3-1 through 3-6 as modified by the following added provisions:

8.2. Beam-to-Column Joints

8.2.a. The required flexural strength, M_u, of each beam-to-column joint shall be the lesser of the following quantities:

1. The plastic bending moment, M_p, of the beam.
2. The moment resulting from the panel zone nominal shear strength, V_n, as determined using Equation 8-1.

The joint is not required to develop either of the strengths defined above if it is shown that under an amplified frame deformation produced by Load Combinations 3-7 and 3-8, the design strength of the members at the connection is adequate to support the vertical loads, and the required lateral force resistance is provided by other means.

8.2.b. The required shear strength, V_u, of a beam-to-column joint shall be determined using the Load Combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from M_u, as defined in Sect. 8.2.a., on each end of the beam. Alternatively, V_u shall be justified by a rational analysis. The required shear strength is not required to exceed the shear resulting from Load Combination 3-7.

8.2.c. The design strength, ϕR_n, of a beam-to-column joint shall be considered adequate to develop the required flexural strength, M_u, of the beam if it conforms to the following:

1. The beam flanges are welded to the column using complete penetration welded joints.
2. The beam web joint has a design shear strength ϕV_n greater than the required shear, V_u, and conforms to either:
   a. Where the nominal flexural strength of the beam, M_n, considering only the flanges is greater than 70 percent of the nominal flexural strength of the entire beam section [i.e., $b_f t_f (d-t_f) F_y \geq 0.7 M_p$]; the web joint shall be made by means of welding or slip-critical high strength bolting, or;
   b. Where $b_f t_f (d-t_f) F_y < 0.7 M_p$, the web joint shall be made by means of welding the web to the column directly or through shear tabs. That welding shall have a design strength of at least 20 percent of the nominal flexural strength of the beam web. The required beam shear, V_u, shall be resisted by further welding or by slip-critical high-strength bolting or both.

8.2.d. Alternate Joint Configurations: For joint configurations utilizing welds or high-strength bolts, but not conforming to Sect. 8.2.c, the design strength shall be determined by test or calculations to meet the criteria of Sect. 8.2.a. Where conformance is shown by calculation, the design strength of the joint shall be 125 percent of the design strengths of the connected elements.

8.3. Panel Zone of Beam-to-Column Connections (Beam web parallel to column web)

8.3.a. Shear Strength: The required shear strength, V_u, of the panel zone shall be based on beam bending moments determined from the Load Combinations 3-5 and 3-6. However, V_u is not required to exceed the shear forces determined from $0.9\Sigma\phi_b M_p$ of the beams framing into the column flanges at the connection. The design shear strength, $\phi_v V_n$, of the panel zone shall be determined by the following formula:

$$\phi_v V_n = 0.6 \phi_v F_y d_c t_p \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p} \right] \text{ where for this case } \phi_v = 0.75. \qquad (8\text{-}1)$$

where:

t_p = Total thickness of panel zone including doubler plates, in.
d_c = Overall column section depth, in.
b_{cf} = Width of the column flange, in.

t_f = Thickness of the column flange, in.
d_b = Overall beam depth, in.
F_y = Specified yield strength of the panel zone steel, ksi.

8.3.b. Panel Zone Thickness: The panel zone thickness, t_z, shall conform to the following:

$$t_z \geq (d_z + w_z)/90 \tag{8-2}$$

where:

d_z = the panel zone depth between continuity plates, in.
w_z = the panel zone width between column flanges, in.

For this purpose, t_z shall not include any doubler plate thickness unless the doubler plate is connected to the web with plug welds adequate to prevent local buckling of the plate.

Where a doubler plate is used without plug welds to the column web, the doubler plate shall conform to Eq. 8-2.

8.3.c. Panel Zone Doubler Plates: Doubler plates provided to increase the design strength of the panel zone or to reduce the web depth thickness ratio shall be placed next to the column web and welded across the plate width along the top and bottom with at least a minimum fillet weld. The doubler plates shall be fastened to the column flanges using either butt or fillet welded joints to develop the design shear strength of the doubler plate.

8.4. Beam and Column Limitations

8.4.a. Beam Flange Area: There shall be no abrupt changes in beam flange areas in plastic hinge regions.

8.4.b. Width-Thickness Ratios: Beams and columns shall comply with λ_p in Table 8-1 in lieu of those in Table B5.1 of the *Specification.*

8.5. Continuity Plates

Continuity plates shall be provided if required by the provisions in the *Specification* for webs and flanges with concentrated forces and if the nominal column local flange bending strength R_n is less than $1.8F_{yb}b_f t_{bf}$, where:

$R_n = 6.25(t_{cf})^2 F_{yf}$, and
F_{yb} = Specified minimum yield strength of beam, ksi.
F_{yf} = Specified minimum yield strength of column flange, ksi.
b_f = Beam flange width, in.
t_{bf} = Beam flange thickness, in.
t_{cf} = Column flange thickness, in.

Continuity plates shall be fastened by welds to both the column flanges and either the column webs or doubler plates.

8.6. Column-Beam Moment Ratio

At any beam-to-column connection, one of the following relationships shall be satisfied:

TABLE 8-1
Limiting Width Thickness Ratios λ_p for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios λ_p
Flanges of I-shaped non-hybrid sections and channels in flexure. Flanges of I-shaped hybrid beams in flexure.	b/t	$52/\sqrt{F_y}$
Webs in combined flexural and axial compression.	h/t_w	For $P_u/\phi_b P_y \leq 0.125$
		$\frac{520}{\sqrt{F_y}}\left[1 - \frac{1.54P_u}{\phi_b P_y}\right]$
		For $P_u/\phi_b P_y > 0.125$
		$\frac{191}{\sqrt{F_y}}\left[2.33 - \frac{P_u}{\phi_b P_y}\right] \geq \frac{253}{\sqrt{F_y}}$

$$\frac{\Sigma Z_c(F_{yc} - P_{uc}/A_g)}{\Sigma Z_b F_{yb}} \geq 1.0, \tag{8-3}$$

$$\frac{\Sigma Z_c(F_{yc} - P_{uc}/A_g)}{V_n d_b H/(H - d_b)} \geq 1.0, \tag{8-4}$$

where:

A_g = Gross area of a column, in.^{2}
F_{yb} = Specified minimum yield strength of a beam, ksi.
F_{yc} = Specified minimum yield strength of a column, ksi.
H = Average of the story heights above and below the joint, in.
P_{uc} = Required axial strength in the column (in compression) ≥ 0
V_n = Nominal strength of the panel zone as determined from Equation 8-1, ksi.
Z_b = Plastic section modulus of a beam, in.^{3}
Z_c = Plastic section modulus of a column, in.^{3}
d_b = Average overall depth of beams framing into the connection, in.

These requirements do not apply in any of the following cases, provided the columns conform to the requirements of Sect. 8.4:

8.6.a. Columns with $P_{uc} < 0.3F_{yc}A_g$.

8.6.b. Columns in any story that has a ratio of design shear strength to design force 50 percent greater than the story above.

8.6.c. Any column not included in the design to resist the required seismic shears, but included in the design to resist axial overturning forces.

8.7. Beam-to-Column Connection Restraint

8.7.a. Restrained Connection:

1. Column flanges at a beam-to-column connection require lateral support only at the level of the top flanges of the beams when a column is shown to remain elastic outside of the panel zone, using one of the following conditions:

   a. Ratios calculated using Eqs. 8-3 or 8-4 are greater than 1.25.

   b. Column remains elastic when loaded with Load Combination 3-7.

2. When a column cannot be shown to remain elastic outside of the panel zone, the following provisions apply:

   a. The column flanges shall be laterally supported at the levels of both top and bottom beam flanges.

   b. Each column flange lateral support shall be designed for a required strength equal to 2.0 percent of the nominal beam flange strength ($F_y b_f t_f$).

   c. Column flanges shall be laterally supported either directly, or indirectly, by means of the column web or beam flanges.

8.7.b. Unrestrained Connections: A column containing a beam-to-column connection with no lateral support transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral supports as the column height for buckling transverse to the seismic frame and conform to Sect. H of the *Specification* except that:

1. The required column strength shall be determined from the Load Combination 3-5 where E is the least of:

   a. The amplified earthquake force $0.4R \times E$ (where the term $0.4R$ shall be equal to or greater than 1.0).

   b. 125 percent of the frame design strength based on either beam or panel zone design strengths.

2. The L/r for these columns shall not exceed 60.

3. The required column moment transverse to the seismic frame shall include that caused by the beam flange force specified in Sect. 8.7.a.2.b plus the added second order moment due to the resulting column displacement in this direction.

8.8. Lateral Support of Beams

Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral supports shall not exceed $2{,}500\ r_y/F_y$. In addition, lateral supports shall be placed at concentrated loads where an analysis indicates a hinge will be formed during inelastic deformations of the SMF.

9. REQUIREMENTS FOR CONCENTRICALLY BRACED (CBF) BUILDINGS

9.1. Scope

Concentrically Braced Frames (CBF) are braced systems whose worklines essentially intersect at points. Minor eccentricities, where the worklines intersect within the width of the bracing members, are acceptable if accounted for in the design. CBF shall have a design strength as provided in the *Specification* to resist the Load Combinations 3-1 through 3-6 as modified by the following added provisions:

9.2. Bracing Members

9.2.a. Slenderness: Bracing members shall have an $\frac{L}{r} \le \frac{720}{\sqrt{F_y}}$ except as permitted in Sect. 9.5.

9.2.b. Compressive Design Strength: The design strength of a bracing member in axial compression shall not exceed $0.8\phi_c P_n$.

9.2.c. Lateral Force Distribution: Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force shall be resisted by tension braces, unless the nominal strength, P_n, of each brace in compression is larger than the required strength, P_u, resulting from the application of the Load Combinations 3-7 or 3-8. A line of bracing, for the purpose of this provision, is defined as a single line or parallel lines whose plan offset is 10 percent or less of the building dimension perpendicular to the line of bracing.

9.2.d. Width-Thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements in braces shall comply with Sect. B5 in the *Specification*. Braces shall be compact or non-compact, but not slender (i.e., $\lambda < \lambda_r$). Circular sections shall have an outside diameter to wall thickness ratio not exceeding $1{,}300/F_y$; rectangular tubes shall have a flat-width to wall thickness not exceeding $110/\sqrt{F_y}$, unless the circular section or tube walls are stiffened.

9.2.e. Built-up Member Stitches: For all built-up braces, the first bolted or welded stitch on each side of the midlength of a built up member shall be designed to transmit a force equal to 50 percent of the nominal strength of one element to the adjacent element. Not less than two stitches shall be equally spaced about the member centerline.

9.3. Bracing Connections

9.3.a. Forces: The required strength of bracing joints (including beam-to-column joints if part of the bracing system) shall be the least of the following:

1. The design axial tension strength of the bracing member.

2. The force in the brace resulting from the Load Combinations 3-7 or 3-8.

3. The maximum force, indicated by an analysis, that is transferred to the brace by the system.

9.3.b. Net Area: In bolted brace joints, the minimum ratio of effective net section area to gross section area shall be limited by:

$$\frac{A_e}{A_g} \geq \frac{1.2\alpha P_u^*}{\phi_t P_n} \qquad (9\text{-}1)$$

where:

A_e = Effective net area as defined in Equation B3-1 of the *Specification.*
P_u^* = Required strength on the brace as determined in Sect. 9.3.a.
P_n = Nominal tension strength as specified in Chapter D of the *Specification.*
ϕ_t = Special resistance factor for tension = 0.75.
α = Fraction of the member force from Sect. 9.3.a that is transferred across a particular net section.

9.3.c. Gusset Plates:

1. Where analysis indicates that braces buckle in the plane of the gusset plates, the gusset and other parts of the connection shall have a design strength equal to or greater than the in-plane nominal bending strength of the brace.

2. Where the critical buckling strength is out-of-plane of the gusset plate, the brace shall terminate on the gusset a minimum of two times the gusset thickness from the theoretical line of bending which is unrestrained by the column or beam joints. The gusset plate shall have a required compressive strength to resist the compressive design strength of the brace member without local buckling of the gusset plate. For braces designed for axial load only, the bolts or welds shall be designed to transmit the brace forces along the centroids of the brace elements.

9.4. Special Bracing Configuration Requirements

9.4.a. V and Inverted V Type Bracing:

1. The design strength of the brace members shall be at least 1.5 times the required strength using Load Combinations 3-5 and 3-6.

2. The beam intersected by braces shall be continuous between columns.

3. A beam intersected by V braces shall be capable of supporting all tributary dead and live loads assuming the bracing is not present.

4. The top and bottom flanges of the beam at the point of intersection of V braces shall be designed to support a lateral force equal to 1.5 percent of the nominal beam flange strength ($F_y b_f t_f$).

9.4.b. K bracing, where permitted:

1. The design strength of K brace members shall be at least 1.5 times the required strength using Load Combinations 3-5 and 3-6.

2. A column intersected by K braces shall be continuous between beams.

3. A column intersected by K braces shall be capable of supporting all dead and live loads assuming the bracing is not present.

4. Both flanges of the column at the point of intersection of K braces shall be designed to support a lateral force equal to 1.5 percent of the nominal column flange strength ($F_y b_f t_f$).

9.5. Low Buildings

Braced frames not meeting the requirements of Sect. 9.2 through 9.4 shall only be used in buildings not over two stories and in roof structures if Load Combinations 3-7 and 3-8 are used for determining the required strength of the members and connections.

10. REQUIREMENTS FOR ECCENTRICALLY BRACED FRAMES (EBF)

10.1. Scope

Eccentrically braced frames shall be designed so that under inelastic earthquake deformations, yielding will occur in the links. The diagonal braces, the columns, and the beam segments outside of the links shall be designed to remain elastic under the maximum forces that will be generated by the fully yielded and strain hardened links, except where permitted by this section.

10.2. Links

10.2.a. Beams with links shall comply with the width-thickness ratios in Table 8-1.

10.2.b. The specified minimum yield stress of steel used for links shall not exceed F_y = 50 ksi.

10.2.c. The web of a link shall be single thickness without doubler plate reinforcement and without openings.

10.2.d. Except as limited by Sect. 10.2.f., the required shear strength of the link, V_u, shall not exceed the design shear strength of the link, ϕV_n, where:

ϕV_n = Link design shear strength of the link = the lesser of ϕV_p or $2\phi M_p / e$, kips.
$V_p = 0.6F_y(d - 2t_f)\,t_w$, kips.
$\phi = 0.9$.
e = link length, in.

10.2.e. If the required axial strength, P_u, in a link is equal to or less than $0.15P_y$, where $P_y = A_g F_y$, the effect of axial force on the link design shear strength need not be considered.

10.2.f. If the required axial strength, P_u, in a link exceeds $0.15P_y$, the following additional limitations shall be required:

1. The link design shear strength shall be the lesser of ϕV_{pa} or $2\phi M_{pa} / e$, where:

$$V_{pa} = V_p\sqrt{1 - (P_u / P_y)^2}$$

$M_{pa} = 1.18M_p[1 - (P_u / P_y)]$
$\phi = 0.9$

2. The length of the link shall not exceed:

$$[1.15 - 0.5\rho(A_w / A_g)]1.6M_p / V_p \text{ for } \rho(A_w / A_g) \geq 0.3 \text{ and}$$

$$1.6M_p / V_p \text{ for } \rho(A_w / A_g) < 0.3, \text{ where:}$$

$A_w = (d - 2t_f)\, t_w$
$\rho = P_u / V_u$

10.2.g. The link rotation angle is the plastic angle between the link and the beam outside of the link when the total story drift is 0.4*R* times the drift determined using the specified base shear *V*. The term 0.4*R* shall be equal to or greater than 1.0. Except as noted in Sect. 10.4.d, the link rotation angle shall not exceed the following values:

1. 0.09 radians for links of length $1.6M_p / V_p$ or less.
2. 0.03 radians for links of length $2.6M_p / V_p$ or greater.
3. Linear interpolation shall be used for links of length between $1.6M_p / V_p$ and $2.6M_p / V_p$.

10.2.h. Alternatively, the top story of an EBF building having over five stories shall be a CBF.

10.3. Link Stiffeners

10.3.a. 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 $0.75t_w$ or ⅜-in., whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

10.3.b. Links shall be provided with intermediate web stiffeners as follows:

1. Links of lengths $1.6M_p / V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.09 radians or $(52t_w - d/5)$ for link rotation angles of 0.03 radians or less. Linear interpolation shall be used for values between 0.03 and 0.09 radians.
2. Links of length greater than $2.6M_p / V_p$ and less than $5M_p / V_p$ shall be provided with intermediate web stiffeners placed at a distance of $1.5b_f$ from each end of the link.
3. Links of length between $1.6M_p / V_p$ and $2.6M_p / V_p$ shall be provided with intermediate web stiffeners meeting the requirements of 1 and 2 above.
4. No intermediate web stiffeners are required in links of lengths greater than $5M_p / V_p$.
5. Intermediate link web stiffeners shall be full depth. For links less than 25 inches in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or ⅜-in., whichever is larger, and the width shall not less than $(b_f/2) - t_w$. For links 25 inches in depth or greater, similar intermediate stiffeners are required on both sides of the web.

10.3.c. Fillet welds connecting link stiffener to the link web shall have a design strength adequate to resist a force of $A_{st}F_y$, in which A_{st} = area of the stiffener. The design strength of fillet welds fastening the stiffener to the flanges shall be adequate to resist a force of $A_{st}F_y / 4$.

10.4. Link-to-Column Connections

Where a link is connected to a column, the following additional requirements shall be met:

10.4.a. The length of links connected to columns shall not exceed $1.6M_p / V_p$ unless it is demonstrated that the link-to-column connection is adequate to develop the required inelastic rotation of the link.

10.4.b. The link flanges shall have complete penetration welded joints to the column. The joint of the link web to the column shall be welded. The required strength of the welded joint shall be at least the nominal axial, shear, and flexural strengths of the link web.

10.4.c. The need for continuity plates shall be determined according to the requirements of Sect. 8.5.

10.4.d. Where the link is connected to the column web, the link flanges shall have complete penetration welded joints to plates and the web joint shall be welded. The required strength of the link web shall be at least the nominal axial, shear, and flexural strength of the link web. The link rotation angle shall not exceed 0.015 radians for any link length.

10.5. Lateral Support of Link

Lateral supports shall be provided at both the top and bottom flanges of link at the ends of the link. End lateral supports of links shall have a design strength of 6 percent of the link flange nominal strength computed as $F_y b_f t_f$.

10.6. Diagonal Brace and Beam Outside of Link

10.6.a. The required combined axial and moment strength of the diagonal brace shall be the axial forces and moments generated by 1.25 times the nominal shear strength of the link as defined in Sect. 10.2. The design strengths of the diagonal brace, as determined by Sect. H (including Appendix H) of the *Specification*, shall exceed the required strengths as defined above.

10.6.b. The required strength of the beam outside of the link shall be the forces generated by at least 1.25 times the nominal shear strength of the link and shall be provided with lateral support to maintain the stability of the beam. Lateral supports shall be provided at both top and bottom flanges of the beam and each shall have a design strength to resist at least 1.5 percent of the beam flange nominal strength computed as $F_y b_f t_f$.

10.6.c. At the connection between the diagonal brace and the beam at the link end of the brace, the intersection of the brace and beam centerlines shall be... the end of the link or in the link. The beam shall not be spliced within or adjacent to the connection between the beam and the brace.

10.6.d. The required strength of the diagonal brace-to-beam connection at the link end of the brace shall be at least the nominal strength of the brace. No part of this connection shall extend over the link length. If the brace resists a portion of the link end moment, the connection shall be designed as Type FR (Fully Restrained).

10.6.e. The width-thickness ratio of brace shall satisfy λ_p of Table B5.1 of the *Specification.*

10.7. Beam-to-Column Connections

Beam-to-column connections away from links are permitted to be designed as a pin in the plane of the web. The connection shall have a design strength to resist torsion about the longitudinal axis of the beam based on two equal and opposite forces of at least 1.5 percent of the beam flange nominal strength computed as $F_y b_f t_f$ acting laterally on the beam flanges.

10.8. Required Column Strength

The required strength of columns shall be determined by Load Combinations 3-5 and 3-6 except that the moments and axial loads introduced into the column at the connection of a link or brace shall not be less than those generated by 1.25 times the nominal strength of the link.

11. QUALITY ASSURANCE

The general requirements and responsibilities for performance of a quality assurance plan shall be in accordance with the requirements of the regulatory agency and specifications by the design engineer.

The special inspections and special tests needed to establish that the construction is in conformance with these provisions shall be included in a quality assurance plan.

The minimum special inspection and testing contained in the quality assurance plan beyond that required by the *Specification* shall be as follows:

Groove welded joints subjected to net tensile forces which are part of the seismic force resisting systems of Sects. 8, 9, and 10 shall be tested 100 percent either by ultrasonic testing or by other approved equivalent methods conforming to AWS D1.1.

Exception: The nondestructive testing rate for an individual welder shall be reduced to 25 percent with the concurrence of the person responsible for structural design, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder.

Part II—Allowable Stress Design (ASD) Alternative

As an alternative to the LRFD seismic design procedures for structural steel design given in PART I, the design procedures in the *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design,* AISC 1989 are permitted as modified by PART II of these provisions. When using ASD, the provisions of PART I of these seismic provisions shall apply except the following sections shall be substituted for, or added to, the appropriate sections as indicated:

1. SCOPE

Revise the first paragraph of PART I, Sect. 1 to read as follows:

These special requirements are to be applied in conjunction with the AISC *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* hereinafter referred to as *Specification.* They are intended for the design and construction of structural steel members and connections in buildings for which the design forces resulting from earthquake motions have been determined on the basis of energy dissipation in the nonlinear range of response.

3. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

Substitute the following for Section 3.2 in PART I:

3.2. Nominal Strengths

The nominal strengths of members shall be determined as follows:

3.2.a. Replace Sect. A5.2 of the *Specification* to read: "The nominal strength of structural steel members for resisting seismic forces acting alone or in combination with dead and live loads shall be determined by multiplying 1.7 times the allowable stresses in Sect. D, E, F, G, J, and K."

3.2.b. Amend the first paragraph of Sect. N1 of the *Specification* by deleting "or earthquake" and adding: "The nominal strength of members shall be determined by the requirements contained herein. Except as modified by these rules, all pertinent provisions of Chapters A through M shall govern."

3.2.c. In Sect H1 of the *Specification* the definition of F_e' shall read as follows:

$$F_e' = \frac{\pi^2 E}{(K l_b / r_b)^2}$$

where:

l_b = the actual length in the plane of bending.
r_b = the corresponding radius of gyration.
K = the effective length factor in the plane of bending.

Add the following section to PART I:

3.3. Design Strengths

3.3.a. The design strengths of structural steel members and connections subjected to seismic forces in combination with other prescribed loads shall be determined by converting allowable stresses into nominal strengths and multiplying such nominal strengths by the resistance factors herein.

3.3.b. Resistance factors, ϕ, for use in Part II shall be as follows:

Flexure	$\phi_b = 0.90$
Compression and axially loaded composite members	$\phi_c = 0.85$
Eyebars and pin connected members:	
Shear of the effective area	$\phi_{sf} = 0.75$
Tension on net effective area	$\phi_t = 0.75$
Bearing on the project area of pin	$\phi_t = 1.0$
Tension members:	
Yielding on gross section	$\phi_t = 0.90$
Fracture in the net section	$\phi_t = 0.75$
Shear	$\phi_v = 0.90$
Connections:	
Base plates that develop the strength of the members or structural systems	$\phi = 0.90$
Welded connections that do not develop the strength of the member or structural system, including connection of base plates and anchor bolts	$\phi = 0.67$
Partial Penetration welds in columns when subjected to tension stresses	$\phi = 0.80$
High strength bolts (A325 and A490) and rivets:	
Tensile strength	$\phi = 0.75$
Shear strength in bearing-type joints	$\phi = 0.65$
Slip-critical joints	$\phi = 1.0$
A307 bolts:	
Tensile strength	$\phi = 0.75$
Shear strength in bearing-type joints	$\phi = 0.60$

Substitute the following for Section 7 in PART I in its entirety:

7. REQUIREMENTS FOR ORDINARY MOMENT FRAMES (OMF)

7.1. Scope

Ordinary Moment Frames (OMF) shall have a design strength as provided in the *Specification* to resist the Load Combinations 3-5 and 3-6 as modified by the following added provisions:

7.2. Joint Requirements

All beam-to-column and column to beam connections in OMF which resist seismic forces shall meet one of the following requirements:

7.2.a. Type 1 connections conforming with Sect. 8.2, except that the required flexural strength, M_u, of a column-to-beam joint are not required to exceed that required to develop the nominal plastic flexural strength of the connection.

7.2.b. Type 1 connections capable of inelastic deformation and the design strengths of the connections meeting the requirements of Sect. 7.1 using the Load Combinations 3-7 and 3-8.

7.2.c. Either Type 1 or Type 3 connections are permitted provided:

1. The design strengths of the members and connections meet the requirements of Sect. 7.1.
2. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at a story drift calculated at a horizontal load of $0.4R \times E$ (where the term $0.4R$ is equal to or greater than 1.0).
3. The additional drift due to Type 3 connections shall be considered in design.

Type 1 and Type 3 connections are described in detail in Sect. A2 of the *Specification.*

Substitute the following in Sections 10.6.a and 10.6.d in PART I:

10.6.a. Delete reference to Appendix H.

10.6.d. The last sentence shall read: "If the brace resists a portion of the link end moment as described above, the connection shall be designed as a Type 1 connection."