
Seismic Provisions for Structural Steel Buildings

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PREFACE

The AISC *Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings* is intended to cover the common design criteria in routine office practice. Accordingly, it is not feasible to also cover the many special and unique problems encountered within the full range of structural design practice. This AISC *Seismic Provisions for Structural Steel Buildings* is a separate document that addresses one such topic: the design and construction of structural steel and composite structural steel/reinforced concrete building systems in seismic regions. These Provisions are in three parts: Part I is intended for the design and construction of structural steel buildings; Part II is intended for the design and construction of composite structural steel/reinforced concrete buildings; Part III is an allowable stress design alternative to the LRFD provisions for structural steel buildings in Part I. Additionally, a list of Symbols, a Glossary, and a non-mandatory Commentary with background information are provided. The first letter(s) of words or terms that appear in the glossary are generally capitalized throughout these Provisions.

The AISC Committee on Specification, Task Committee 113—Seismic Provisions is responsible for ongoing development of these Provisions. Additionally, the AISC Committee on Specification has enhanced these Provisions through careful scrutiny, discussion, suggestion for improvements, and endorsement. AISC further acknowledges the various contributions of several groups to the completion of this document: the Building Seismic Safety Council (BSSC), the National Science Foundation (NSF), the SAC Joint Venture, and the Structural Engineers Association of California (SEAOC).

The principal changes in this revision of the Seismic Provisions are: extensive modifications to Special Moment Frames (SMF), the addition of special requirements for welded and bolted connections, an expanded diversity of structural steel systems, such as Intermediate Moment Frames (IMF), Special Truss Moment Frames (STMF), and Special Concentrically Braced Frames (SCBF); the addition of Part II, which covers composite structural steel/reinforced concrete seismic systems; and, the incorporation of Appendix S with provisions for the evaluation of moment connection performance through testing.

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Symbols

Numbers in parentheses after the definition of a symbol refer to the Section in either Part I or II of these Provisions in which the symbol is first used.

A_f	Flange area, in. ² (I-8)
A_g	Gross area, in. ² (I-9)
A_s	Cross-sectional area of structural steel elements in composite members, in. ² (II-6)
A_s/A_g	Ratio of cross-sectional area of structural steel to the gross area of a composite column. (II-6)
A_{sh}	Minimum area of tie reinforcement, in. ² (II-6)
A_{sp}	Horizontal area of the steel plate in composite shear wall, in. ² (II-5)
A_{st}	Area of Link stiffener, in. ² (I-15)
A_w	Link web area, in. ² (I-15)
D	Dead load due to the weight of the structural elements and permanent features on the building, kips. (I-4) Outside diameter of round HSS, in. (Table I-9-1)
E	Effect of horizontal and vertical earthquake-induced loads. (I-4) The modulus of elasticity of steel, ksi. (I-6)
EI	Flexural elastic stiffness of the chord members of the special segment kip-in. ² (I-12)
F_y	Specified minimum yield stress of the type of steel to be used, ksi. As used in the LRFD Specification, “yield stress” denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have yield point). (I-5)
F_{yb}	F_y of a beam, ksi. (I-9)
F_{yc}	F_y of a column, ksi. (I-9)
F_{ye}	Expected Yield Strength of steel to be used, ksi. (I-6)
F_{yf}	F_y of column flange, ksi.
F_{yh}	Specified minimum yield strength of transverse reinforcement, ksi. (II-6)
F_{yw}	F_y of the panel-zone steel, ksi.
F_u	Specified minimum tensile strength, ksi. (I-7)
H	Average story height above and below a beam-to-column connection, in. (I-15)
K	Effective length factor for prismatic member. (I-13)
L	Live load due to occupancy and moveable equipment, kips. (I-4) Span length of the truss, in. (I-12) Unbraced length of compression or bracing member, in. (I-13)
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, in. (I-12)
L_s	Length of the special segment, in. (I-12)
M_{nc}	Nominal flexural strength of the chord member of the special segment, kip in. (I-12)

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M_p	Nominal plastic flexural strength, kip-in. (I-9)
M_{pa}	Nominal plastic flexural strength modified by axial load, kip-in. (I-15)
M_{pe}	Nominal plastic flexural strength using Expected Yield Strength of steel, kip-in. (I-8)
M_u	Required flexural strength on a member or joint, kip-in. (I-8)
$P-\Delta$	Second order effect of column axial loads and lateral deflection on moments in members, kip-in. (I-9)
P_n	Nominal axial strength of a column, kips. (I-8)
	Nominal axial strength of a composite column, kips. (II-6)
P_{nc}	Nominal axial compressive strength of diagonal members of the special segment, kips. (I-12)
P_{nt}	Nominal axial tensile strength of diagonal members of the special segment, kips. (I-12)
P_o	Nominal axial strength of a composite column at zero eccentricity, kips. (II-5)
P_u	Required axial strength on a column or a Link, kips. (I-8)
	Required axial strength of a composite column, kips. (II-5)
P_{uc}	Required axial strength on a column in compression, kips. (I-9)
P_y	Nominal axial yield strength of a member, which is equal to $F_y A_g$, kips. (I-9)
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces, kips. (I-13)
Q_E	Effect of horizontal seismic forces produced by the base shear, V . (I-4)
R_n	Nominal strength. (I-9)
R_u	Required strength. (I-9)
R_y	Ratio of the Expected Yield Strength F_{ye} to the minimum specified yield strength F_y . (I-5)
S	Snow load, kips. (I-4)
S_a	Design spectral response acceleration. (I-4)
V_n	Nominal shear strength of a member, kips. (I-9)
V_{ns}	Nominal shear strength of the steel plate in a composite plate shear walls, kips. (II-5)
V_p	Nominal shear strength of an active Link, kips. (I-15)
V_{pa}	Nominal shear strength of an active Link modified by the axial load magnitude, kips. (I-15)
V_u	Required shear strength on a member, kips. (I-9)
Y_{con}	Distance from top of steel beam to top of concrete slab or encasement, in. (II-6)
Z	Plastic section modulus of a member, in. ³ (I-9)
α	Angle that diagonal members make with the horizontal. (I-12)
b	Width of compression element as defined in LRFD Specification Section B5.1, in. (Table I-9-1)
b_{cf}	Width of column flange, in. (I-9)
b_f	Flange width, in. (I-9)
b_w	Width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear, in. (II-6)
d	Nominal fastener diameter, in. (I-7)
d_b	Overall beam depth, in. (I-9)
d_c	Overall column depth, in. (I-9)
d_z	Overall panel-zone depth between continuity plates, in. (I-9)
e	EBF Link length, in. (I-15)
f'_c	Specified compressive strength of concrete, ksi. (II-6)

h	Cross-sectional dimension of reinforced concrete or composite column, in. (II-6)
h_c	Assumed web depth for stability, in. (Table I-9-1)
h_{cc}	Cross-sectional dimension of the confined core region in composite columns measured center-to-center of the transverse reinforcement, in. (II-6)
l	unbraced length between stitches of built-up bracing members, in. (I-13)
r	Governing radius of gyration, in. (I-13)
r_y	Radius of gyration about y axis, in. (I-9)
s	Spacing of transverse reinforcement measured along the longitudinal axis of the structural composite member, in. (II-6)
t	Thickness of connected part, in. (I-7)
t_{bf}	Thickness of beam flange, in. (I-9)
t_{cf}	Thickness of column flange, in. (I-9)
t_f	Thickness of flange, in. (Table I-9-1)
t_p	Thickness of panel-zone including doubler plates, in. (I-9)
t_w	Thickness of web, in. (Table I-9-1)
t_z	Thickness of panel-zone (doubler-plate thickness not necessarily included), in. (I-9)
w_z	Width of panel-zone between column flanges, in. (I-9)
z	Minimum plastic section modulus at the Reduced Beam Section, in. ³ (I-9)
Δ	Design story drift, in. (I-6)
ΣM_{pc}^*	Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g , from the top and bottom of the beam moment connection. (I-9)
ΣM_{pb}^*	Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum developed moments from the column face. Maximum developed moments shall be determined from test results. (I-9)
Ω_o	Horizontal seismic overstrength factor. (I-4)
δ	Deformation quantity used to control loading of the Test Specimen. (S6)
δ_y	Value of deformation quantity δ at first significant yield of Test Specimen. (S6)
ρ'	Ratio of required axial force P_u to required shear strength V_u of a Link. (I-15)
λ	Slenderness parameter. (I-13)
λ_p	Limiting slenderness parameter for compact element. (Table I-9-1)
λ_r	Limiting slenderness parameter for non-compact element. (I-14)
ϕ	Resistance factor. (I-8)
ϕ_c	Resistance factor for compression. (I-13)
ϕ_v	Resistance factor for shear strength of panel-zone of beam-to-column connections. (I-9)
ϕ_v	Resistance factor for the shear strength of a composite column. (II-6)
ρ_v	Ratio of distributed vertical or horizontal reinforcement to the gross wall area. (II-5)

Part I

Structural Steel Buildings

Part I Glossary

Applicable Building Code. The building code under which the building is designed.

Beam. A structural member that primarily functions to carry loads transverse to its longitudinal axis; usually a horizontal member in a seismic frame system.

Braced Frame. A vertical truss system of concentric or eccentric type that resists lateral forces on the structural system.

Connection. A combination of joints used to transmit forces between two or more members. Connections are categorized by the type and amount of force transferred (moment, shear, end reaction).

Continuity Plates. Column stiffeners at the top and bottom of the panel-zone; also known as transverse stiffeners.

Design Earthquake. The earthquake represented by the Design Response Spectrum as specified in the Applicable Building Code.

Design Story Drift. The amplified story drift determined as specified in the Applicable Building Code.

Design Strength. Resistance (force, moment, stress, as appropriate) provided by element or connection; the product of the nominal strength and the resistance factor.

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

Dual System. A structural system with the following features: (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment resisting frames (SMF, IMF or OMF) that are capable of resisting at least 25 percent of the base shear and concrete or steel shear walls or steel braced frames (EBF, SCBF or OCBF); and, (3) each system designed to resist the total lateral load in proportion to its relative rigidity.

Eccentrically Braced Frame (EBF). A diagonally braced frame meeting the requirements in Section 15 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection.

Expected Yield Strength. The Expected Yield Strength of steel in structural members is related to the Specified Yield Strength by the multiplier R_y .

Fully Restrained (FR). Sufficient rigidity exists in the connection to maintain the angles between intersecting members.

Inelastic Rotation of Beam-to-Column Connection. The total angle change between the column face at the connection and a line connecting the beam inflection point to the column face, less that part of the angle change occurring prior to yield of the beam.

Intermediate Moment Frame (IMF). A moment frame system that meets the requirements in Section 10.

Inverted-V-Braced Frame. See V-Braced Frame

- Joint.** An area where two or more ends, surfaces or edges are attached. Joints are categorized by the type of fastener or weld used and the method of force transfer.
- K-Braced Frame.** An OCBF 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.** A member that is designed to inhibit lateral buckling or lateral-torsional buckling of primary framing members.
- Link.** In EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column. The length of the Link is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.
- Link Intermediate Web Stiffeners.** Vertical web stiffeners placed within the Link in EBF.
- 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 E'/E times the drift derived using the specified base shear V .
- Link Shear Design Strength.** The lesser of the design shear strength of the Link developed from the moment or shear strength of the Link.
- Load and Resistance Factor Design (LRFD).** A method of proportioning structural components (members, connectors, connecting elements, and assemblages) such that no applicable limit state is exceeded when the building is subjected to all appropriate load combinations.
- Moment Frame.** A building frame system in which seismic shear forces are resisted by shear and flexure in members and connections of the frame.
- Nominal loads.** The magnitudes of the loads specified by the Applicable Building Code.
- Nominal strength.** The capacity of a building 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 Concentrically Braced Frame (OCBF).** A diagonally braced frame meeting the requirements in Section 14 in which all members of the bracing system are subjected primarily to axial forces.
- Ordinary Moment Frame (OMF).** A moment frame system that meets the requirements in Section 11.
- P-Delta Effect.** Second-order effect of column axial loads after lateral deflection of the frame on the shears and moments in members.
- Panel-zone.** The web area of the beam-to-column connection delineated by the extension of beam and column flanges through the connection.
- Partially Restrained (PR).** Insufficient rigidity exists in the connection to maintain the angles between intersecting members.
- Reduced Beam Section.** A ductile reduction in cross-section over a discrete length that promotes a zone of inelasticity in the member.
- Required Strength.** The load effect (force, moment, stress, or as appropriate) acting on a member or connection that is determined by structural analysis from the factored loads using the most appropriate critical load combinations, or as specified in these Provisions.
- Resistance Factor.** A factor that accounts for unavoidable deviations in the actual strength of a member or connection from the Nominal Strength and for the manner and consequences of failure.

Seismic Design Category. A classification assigned to a building based upon such factors as its occupancy and use.

Seismic Force Resisting System. The assembly of structural element in the building that resists seismic forces.

Slip-critical Joint. A bolted joint in which slip resistance on the faying surface(s) of the connection is required.

Special Concentrically Braced Frame (SCBF). A diagonally braced frame meeting the requirements in Section 12 in which all members of the bracing system are subjected primarily to axial forces.

Special Moment Frame (SMF). A moment frame system that meets the requirements in Section 9.

Special Truss Moment Frame (STMF). A truss moment frame system that meets the requirements in Section 13.

Static Yield Strength. The strength of a structural member or connection that is determined on the basis of testing that is conducted under slow monotonic loading until failure.

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

V-Braced Frame. A concentrically braced frame (SCBF or OCBF) 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 (OCBF) 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.

1. SCOPE

These Provisions are intended for the design and construction of structural steel members and connections in the Seismic Force Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response. These Provisions shall apply to buildings that are classified in the Applicable Building Code as Seismic Design Category D (or equivalent) and higher or when required by the Engineer of Record.

These Provisions shall be applied in conjunction with the AISC *Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings*, hereinafter referred to as the LRFD Specification. All members and connections in the Seismic Force Resisting System shall have a design strength as provided in the LRFD Specification to resist Load Combinations A4-1 through A4-6 and shall meet the requirements in these Provisions.

Part I includes a Glossary, which is specifically applicable to this Part, and Appendix S.

2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The documents referenced in these Provisions shall include those listed in LRFD Specification Section A6 with the following additions and modifications:

American Institute of Steel Construction
Load and Resistance Factor Design Specification for Structural Steel Buildings, December 1, 1993
Specification for the Design of Steel Hollow Structural Sections, April 15, 1997

American Society of Civil Engineers
ASCE 7-95

American Society for Testing and Materials		
ASTM A6-96b	ASTM A500-93	ASTM A673-95
ASTM A36-96	ASTM A501-93	ASTM A913-95a
ASTM A53-96	ASTM A572-94c	
ASTM A283-93a	ASTM A588-94	

American Welding Society
AWS D1.1-96

Research Council on Structural Connections
Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts, June 3, 1994

3. SEISMIC DESIGN CATEGORIES

Seismic provisions, the required strength for Seismic Design Categories, Seismic Use Groups or Seismic Zones and the limitations on height and irregularity shall be as specified in the Applicable Building Code; or, when no building code is applicable, as dictated by the conditions involved.

TABLE I-4-1
System Overstrength Factor, Ω_o

Seismic Force Resisting System	Ω_o
All moment-frame systems meeting Part I requirements	3
Eccentrically Braced Frames (EBF) meeting Part I requirements	$2\frac{1}{2}$
All other systems meeting Part I requirements	2

4. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

4.1. Loads and Load Combinations

The loads and load combinations shall be those in LRFD Specification Section A4.1, except as modified throughout these Provisions.

Q_E is the horizontal component of the earthquake load E required in the Applicable Building Code. Where required in these Provisions, an amplified horizontal earthquake load $\Omega_o Q_E$ shall be used in lieu of Q_E as given in the load combinations below. The term Ω_o is the System Overstrength Factor as defined in the Applicable Building Code. In the absence of such definition, Ω_o shall be as listed in Table I-4-1.

The additional load combinations using the amplified horizontal earthquake load are:

$$1.2D + 0.5L + 0.2S + \Omega_o Q_E \quad (4-1)$$

$$0.9D - \Omega_o Q_E \quad (4-2)$$

Exception: The load factor on L in Load Combination 4-1 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.

Orthogonal earthquake effects shall be included in the analysis as required in the Applicable Building Code, except that, when consideration of the load $\Omega_o Q_E$ is required, orthogonal earthquake effects need not be included.

4.2. Nominal Strength

The nominal strength of systems, members and connections shall meet the requirements in the LRFD Specification, except as modified throughout these Provisions.

5. STORY DRIFT

The Design Story Drift and story drift limits shall be determined as specified in the Applicable Building Code.

6. MATERIALS

6.1. Material Specifications

Structural steel used in the Seismic Force Resisting System shall meet the requirements in LRFD Specification Section A3.1a, except as modified in this

Section. For buildings over one story in height, the steel used in the Seismic Force Resisting Systems described in Sections 9, 10, 11, 12, 13, 14 and 15 shall meet one of the following ASTM Specifications: A36, A53, A500 (Grade B or C), A501, A572 (Grade 42 or 50), A588 or A913 (Grade 50 or 65). The steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283 Grade D. The specified minimum yield strength of steel to be used for members in which inelastic behavior is expected under Load Combinations 4-1 and 4-2 shall not exceed 50 ksi unless the suitability of the material is determined by testing or other rational criteria. This limitation does not apply to columns for which the only expected inelastic behavior is yielding at the column base.

6.2. Material Properties for Determination of Required Strength for Connections or Related Members

When required in these Provisions, the required strength of a connection or related member shall be determined from the Expected Yield Strength F_{ye} of the connected member, where

$$F_{ye} = R_y F_y \quad (6-1)$$

F_y is the specified minimum yield strength of the grade of steel to be used. For rolled shapes and bars, R_y shall be taken as 1.5 for ASTM A36 and 1.3 for A572 Grade 42. For rolled shapes and bars of other grades of steel and for plates, R_y shall be taken as 1.1. Other values of R_y are permitted to be used if the value of F_{ye} is determined by testing that is conducted in accordance with the requirements for the specified grade of steel.

6.3. Notch-Tough Steel

When they are used as members in the Seismic Force Resisting System, ASTM A6 Group 3 shapes with flanges 1½-in. thick and thicker, ASTM A6 Groups 4 and 5 shapes, and plates that are 1½-in. thick or thicker in built-up cross-sections shall have a minimum Charpy V-Notch (CVN) toughness of 20 ft-lbs at 70 degrees F, determined as specified in LRFD Specification Section A3.1c.

7. CONNECTIONS, JOINTS AND FASTENERS

7.1. Scope

Connections, joints and fasteners that are part of the Seismic Force Resisting System shall meet the requirements in LRFD Specification Chapter J, except as modified in this Section.

7.2. Bolted Joints

- 7.2a. All bolts shall be fully tensioned high-strength bolts. All faying surfaces shall be prepared as required for Class A or better slip-critical joints. The design shear strength of bolted joints is permitted to be calculated as that for bearing-type joints.
- 7.2b. Bolted joints shall not be designed to share load in combination with welds on the same faying surface.
- 7.2c. The bearing strength of bolted joints shall be provided using either standard holes or short-slotted holes with the slot perpendicular to the

line of force, unless an alternative hole type is justified as part of a tested assembly; see Appendix S.

- 7.2d. The design strength of bolted joints in shear and/or combined tension and shear shall be determined in accordance with LRFD Specification Sections J3.7 and J3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.
- 7.2e. Bolted connections for members that are a part of the Seismic Force Resisting System shall be configured such that a ductile limit-state either in the connection or in the member controls the design.

7.3. Welded Joints

- 7.3a. Welding shall be performed in accordance with a Welding Procedure Specification (WPS) as required in AWS D1.1 and approved by the Engineer of Record. The WPS variables shall be within the parameters established by the filler-metal manufacturer.
- 7.3b. All complete-joint-penetration groove welds used in the Seismic Force Resisting System shall be made with a filler metal that has a minimum CVN toughness of 20 ft-lbs at minus 20 degrees F, as determined by AWS classification or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these Provisions.
- 7.3c. For members and connections that are part of the Seismic Force Resisting System, discontinuities created by errors or by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging, and flame cutting, shall be repaired as required by the Engineer of Record.

8. COLUMNS

8.1. Scope

Columns in the Seismic Force Resisting System shall meet the requirements in the LRFD Specification and in this Section.

8.2. Column Strength

When $P_u/\phi P_n$ is greater than 0.4, the requirements in Sections 8.2a, 8.2b and 8.2c shall be met.

- 8.2a. The required axial compressive strength, considered in the absence of any applied moment, shall be determined from Load Combination 4-1.
- 8.2b. The required axial tensile strength, considered in the absence of any applied moment, shall be determined from Load Combination 4-2.
- 8.2c. The required strengths determined in Sections 8.2a and 8.2b need not exceed either of the following:
 - a. The maximum load transferred to the column considering $1.1R_y$ times the nominal strengths of the connecting beam or brace elements of the building.
 - b. The limit as determined from the resistance of the foundation to overturning uplift.

8.3. Column Splices

The design strength of column splices shall meet or exceed the required strength determined from Section 8.2.

8.3a. Column splices that are made with fillet welds or partial-joint-penetration groove welds shall not be located within 4 ft nor one-half the column clear height of beam-to-column connections, whichever is less. Welded column splices that are subject to a calculated net tensile stress under Load Combination 4-2 shall be made using filler metal with CVN toughness as required in Section 7.3b and shall meet both of the following requirements:

1. The design strength of partial-joint-penetration groove welded joints shall be at least equal to 200 percent of the required strength.
2. The minimum required strength for each flange shall be 0.5 times $R_y F_y A_f$, where $R_y F_y$ is the Expected Yield Strength of the column material and A_f is the flange area of the smaller column connected.

8.3b. Beveled transitions are not required when changes in thickness and width of flanges and webs occur in column splices where partial-joint-penetration groove welded joints are permitted according to Section 8.3a.

9. SPECIAL MOMENT FRAMES (SMF)

9.1. Scope

Special Moment Frames (SMF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the Design Earthquake. SMF shall meet the requirements in this Section.

9.2. Beam-to-Column Joints and Connections

9.2a. The design of all beam-to-column joints and connections used in the Seismic Force Resisting System shall be based upon qualifying cyclic test results in accordance with Appendix S that demonstrate an inelastic rotation of at least 0.03 radians. Qualifying test results shall consist of at least two cyclic tests and are permitted to be based upon one of the following requirements:

- a. Tests reported in research or documented tests performed for other projects that are demonstrated to reasonably match project conditions.
- b. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes.

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal stresses that are consistent with the tested assemblies and that considers the adverse effects of larger material and weld thickness and variations in material properties. Extrapolation of test results shall be based upon similar combinations of member sizes.

The actual connections shall be constructed using materials, configurations, processes and quality control and assurance methods that match as closely as is practicable those of the tested connections. As a minimum, the quality control and assurance methods shall meet the requirements in Section 16. Beams with a tested yield strength that is more than 15 percent below F_{ye} shall not be used in qualification testing.

- 9.2b.** Beam-to-column connection testing shall demonstrate a flexural strength, determined at the column face, that is at least equal to the nominal plastic moment of the beam M_p at the required inelastic rotation (see Appendix S), except as follows:
- When beam local buckling rather than beam yielding limits the flexural strength of the beam, or when connections incorporating a Reduced Beam Section are used, the minimum flexural strength shall be $0.8M_p$ of the tested beam.
 - Connections that accommodate the required rotations within the connecting elements and maintain the design strength as specified in Section 1 are permitted, provided it can be demonstrated by rational analysis that any additional drift due to connection deformation can be accommodated by the building. Such rational analysis shall include the effects of overall frame stability including second-order effects.
- 9.2c.** The required shear strength V_u of a beam-to-column connection shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from the application of $1.1R_yF_yZ$ in the opposite sense on each end of the beam. Alternatively, a lesser value of V_u is permitted if justified by rational analysis. The required shear strength need not exceed the shear resulting from Load Combination 4-1.

**9.3. Panel-Zone of Beam-to-Column Connections
(beam web parallel to column web)**

- 9.3a.** Shear Strength: The required shear strength R_u of the panel-zone shall be determined by applying Load Combinations 4-1 and 4-2 to the connected beam or beams in the plane of the frame at the column. R_u need not exceed the shear force determined from 0.8 times $\sum R_y M_p$ of the beams framing to the column flanges at the connection. The design shear strength $\phi_v R_v$ of the panel-zone shall be determined using $\phi_v = 0.75$. When $P_u \leq 0.75P_y$,

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_p} \right] \quad (9-1)$$

When $P_u > 0.75P_y$, R_v shall be calculated using LRFD Specification Equation K1-12. In the above equation,

- t_p = total thickness of panel-zone including doubler plate(s), in.
 d_c = overall column depth, in.
 b_{cf} = width of the column flange, in.

t_{cf} = thickness of the column flange, in.
 d_b = overall beam depth, in.
 F_y = specified minimum yield strength of the panel-zone steel, ksi.

- 9.3b.** Panel-Zone Thickness: The individual thicknesses t of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90 \quad (9-2)$$

where

t = thickness of column web or doubler plate, in.
 d_z = panel-zone depth between continuity plates, in.
 w_z = panel-zone width between column flanges, in.

Alternatively, when local buckling of the column web and doubler plate is prevented with plug welds between them, the total panel-zone thickness shall satisfy Equation 9-2.

- 9.3c.** Panel-Zone Doubler Plates: Doubler plates shall be welded to the column flanges using either a complete-joint-penetration groove-welded or fillet-welded joint that develops the design shear strength of the full doubler plate thickness. When doubler plates are placed against the column web, they shall be welded across the top and bottom edges to develop the proportion of the total force that is transmitted to the doubler plate. When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

9.4. Beam and Column Limitations

- 9.4a.** Beam Flange Area: Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is permitted if testing demonstrates that the resulting configuration can develop stable plastic hinges that meet the requirements in Section 9.2b. The Reduced Beam Section shall meet the design strength as specified in Section 1.
- 9.4b.** Width-Thickness Ratios: Beams shall comply with λ_p in Table I-9-1. When the ratio in Equation 9-3 is less than or equal to 1.25, columns shall comply with λ_p in Table I-9-1. Otherwise, columns shall comply with λ_p in LRFD Specification Table B5.1.

9.5. Continuity Plates

Continuity plates shall be provided to match the tested connection.

9.6. Column-Beam Moment Ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0 \quad (9-3)$$

TABLE I-9-1
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 rolled beams, hybrid or welded beams and channels in flexure	b/t	$52/\sqrt{F_y}$
Webs in combined flexural and axial compression	h_c/t_w	For $P_u/\phi_b P_y \leq 0.125$
		$\frac{520}{\sqrt{F_y}} \left[1 - 1.54 \frac{P_u}{\phi_b P_y} \right]$
		For $P_u/\phi_b P_y > 0.125$
		$\frac{191}{\sqrt{F_y}} \left[2.33 - \frac{P_u}{\phi_b P_y} \right] \geq \frac{253}{\sqrt{F_y}}$
Round HSS in axial compression or flexure	D/t	$1300/F_y$
Rectangular HSS in axial compression or flexure	b/t or h_c/t	$110/\sqrt{F_y}$

where

$\sum M_{pc}^*$ = The sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\sum M_{pc}^*$ is determined by summing the projections of the nominal flexural strengths of the column (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. It is permitted to take $\sum M_{pc}^* = \sum Z_c(F_{yc} - P_{uc}/A_g)$. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

$\sum M_{pb}^*$ = The sum of the moment(s) in the beam(s) at the intersection of the beam and column centerlines. $\sum M_{pb}^*$ is determined by summing the projections of the nominal beam flexural strength(s) at the plastic hinge location(s) to the column centerline. It is permitted to take $\sum M_{pb}^* = \sum (1.1R_y M_p + M_v)$, where M_v is the additional moment due to shear amplification from the location of the plastic hinge to the column centerline. Alternatively, it is permitted to determine $\sum M_{pb}^*$ from test results as required in Section 9.2a or by rational analysis based upon the tests. When connections with Reduced Beam Sections are used, it is permitted to take

$\sum M_{pb}^* = \sum (1.1R_y F_y z + M_v)$, where z is the minimum plastic section modulus at the Reduced Beam Section.

- A_g = gross area of column, in.²
- F_{yc} = specified minimum yield strength of column, ksi.
- P_{uc} = required column axial compressive strength, kips (a positive number).
- Z_c = plastic section modulus of the column, in.³

When columns conform to the requirements in Section 9.4, this requirement does not apply in the cases covered in Sections 9.6a and 9.6b:

9.6a. Columns with $P_{uc} < 0.3F_{yc}A_g$ for all load combinations other than those specified in Load Combinations 4-1 and 4-2 that meet either of the following requirements:

1. Columns used in a one-story building or the top story of a multistory building.
2. Columns where: (1) the sum of the design shear strengths of all exempted columns in the story is less than 20 percent of the required story shear strength; and (2) the sum of the design shear strengths of all exempted columns on each column line within that story is less than 33 percent of the required story shear strength 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 percent of the plan dimension perpendicular to the line of columns.

9.6b. Columns in any story that has a ratio of design shear strength to required shear strength that is 50 percent greater than the story above.

9.7. Beam-to-Column Connection Restraint

9.7a. Restrained Connections:

1. Column flanges at beam-to-column connections 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 under either of the following conditions:
 - a. The ratio calculated using Equation 9-3 is greater than 1.25.
 - b. The column remains elastic under Load Combination 4-1.
2. When a column cannot be shown to remain elastic outside of the panel-zone, the following requirements shall apply:
 - a. The column flanges shall be laterally supported at the levels of both the top and bottom beam flanges.
 - b. Each column-flange lateral support shall be designed for a required strength that is equal to 2 percent of the nominal beam flange strength ($F_y b_f t_{bf}$).
 - c. Column flanges shall be laterally supported, either directly or indirectly, by means of the column web or by the flanges of perpendicular beams.

9.7b. 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 shall conform to LRFD Specification Chapter H, except that:

1. The required column strength shall be determined from LRFD Specification Load Combination A4-5, except that E shall be taken as the lesser of:
 - a. The amplified earthquake force $\Omega_o Q_E$.
 - b. 125 percent of the frame design strength based upon either the beam design flexural strength or panel-zone design shear strength.
2. The slenderness L/r for the column shall not exceed 60.
3. The column required flexural strength transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Section 9.7a.2.b in addition to the second-order moment due to the resulting column flange displacement.

9.8. Lateral Support of Beams

Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral supports shall not exceed $2500r_y/F_y$. In addition, lateral supports shall be placed near concentrated forces, changes in cross-section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF.

If members with Reduced Beam Sections, tested in accordance with Appendix S are used, the placement of lateral support for the member shall be consistent with that used in the tests. Any lateral support adjacent to the Reduced Beam Section shall meet the requirements in Section 15.5.

10. INTERMEDIATE MOMENT FRAMES (IMF)

10.1. Scope

Intermediate Moment Frames (IMF) are expected to withstand moderate inelastic deformations when subjected to the forces resulting from the motions of the Design Earthquake. IMF shall meet the requirements of this Section and shall be designed so that the earthquake-induced inelastic deformations are accommodated by the yielding of members of the frame when FR moment connections are used or by yielding of connection elements when PR moment connections are used. FR and PR moment connections are described in LRFD Specification Section A2.2.

IMF shall conform to the requirements for SMF in Section 9 except for the following modifications:

Replace Sections 9.2a and 9.2b with Sections 10.2a and 10.2b as follows:

10.2. Beam-to-Column Joints and Connections

- 10.2a.** The design of all beam-to-column joints and connections used in the Seismic Force Resisting System shall be based upon qualifying cyclic test results in accordance with Appendix S that demonstrate an inelastic rotation of at least 0.02 radians. Qualifying cyclic tests results shall consist of at least two cyclic tests and shall meet the requirements in Section 9.2a.
- 10.2b.** Beam-to-column connection testing shall demonstrate a flexural strength, determined at the column face, that is at least equal to the nominal plastic moment of the beam M_p at the required inelastic rotation (see Appendix S), except as follows:
- When beam local buckling rather than beam yielding limits the flexural strength of the beam, or when connections incorporating a Reduced Beam Section are used, the minimum flexural strength shall be $0.8M_p$ of the tested beam.
 - Connections that accommodate the required rotations within the connection elements and maintain the design strength as specified in Section 1 are permitted, provided it can be demonstrated by rational analysis that any additional drift due to connection deformation can be accommodated by the building. Such rational analysis shall include the effects of overall frame stability including second order effects.

Replace Section 9.4b with 10.4b as follows:

- 10.4b.** Width-Thickness Ratios: Beams shall comply with λ_p in LRFD Specification Table B5.1. When the ratio in Equation 9-3 is less than or equal to 1.25, columns shall comply with λ_p in Table I-9-1. Otherwise, columns shall comply with λ_p in LRFD Specification Table B5.1.

Replace Section 9.8 with 10.8 as follows:

10.8. Lateral Support at Beams

Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral supports shall not exceed $3,600r_y/F_y$. In addition, lateral supports shall be placed near concentrated forces, changes in cross-section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF.

11. ORDINARY MOMENT FRAMES (OMF)

11.1. SCOPE

Ordinary Moment Frames (OMF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the Design Earthquake. OMF shall meet the requirements in this Section.

11.2. Beam-to-Column Joints and Connections

11.2a. Beam-to-column connections shall be made with welds or high-strength bolts. Connections are permitted to be FR or PR moment connections as follows:

1. FR moment connections that are part of the Seismic Force Resisting System shall be designed for a required flexural strength M_u that is at least equal to $1.1R_y M_p$ of the beam or girder or the maximum moment that can be delivered by the system, whichever is less. For connections with welded flange joints, weld backing and run-off tabs shall be removed and repaired including the use of a reinforcing fillet weld, except that the top-flange backing is permitted to remain in place if it is attached to the column flange with a continuous fillet weld on the edge below the complete-joint-penetration groove weld. Partial-joint-penetration groove welds and fillet welds shall not be used to resist tensile forces in the connections.

Alternatively, the design of all beam-to-column joints and connections used in the Seismic Force Resisting System shall be based upon qualifying cyclic test results in accordance with Appendix S that demonstrate an inelastic rotation of at least 0.01 radians. Cyclic test results shall consist of at least two tests and shall be based upon the procedures described in Section 9.2a.

2. PR moment connections are permitted when the following requirements are met:
 1. Such connections shall provide for the design strength as specified in Section 1.
 2. The nominal flexural strength of the connection shall be equal to or exceed 50 percent of M_p of the connected beam or column, whichever is less.
 3. Adequate rotation capacity shall be demonstrated in the connections by cyclic testing at rotations corresponding to the Design Story Drift.
 4. The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.

FR and PR moment connections are described in LRFD Specification Section A2.2.

- 11.2b.** For FR moment connections, the required shear strength V_u of a beam-to-column connection shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from M_u , as defined in Section 11.2a.1. For PR moment connections, V_u shall be determined from the load combination above plus the shear resulting from the maximum end moment that the PR moment connections are capable of resisting.

11.3. Continuity Plates

When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, continuity plates shall

be provided to transmit beam flange forces to the column web or webs. Such plates shall have a minimum thickness equal to that of the beam flange or beam-flange connection plate. The welded joints of the continuity plates to the column flanges shall be made with either complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds combined with reinforcing fillet welds, or two-sided fillet welds and shall provide a design strength that is at least equal to the design strength of the contact area of the plate with the column flange. The welded joints of the continuity plates to the column web shall have a design shear strength that is at least equal to the lesser of the following:

- a. The sum of the design strengths at the connections of the continuity plate to the column flanges.
- b. The design shear strength of the contact area of the plate with the column web.
- c. The weld design strength that develops the design shear strength of the column panel-zone.
- d. The actual force transmitted by the stiffener.

Continuity plates are not required if tested connections demonstrate that the intended inelastic rotation can be achieved without their use.

12. SPECIAL TRUSS MOMENT FRAMES (STMF)

12.1. Scope

Special Truss Moment Frames (STMF) are expected to withstand significant inelastic deformation within a specially designed segment of the truss when subjected to the forces from the motions of the Design Earthquake. STMF shall be limited to span lengths between columns not to exceed 65 ft and overall depth not to exceed 6 ft. The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. STMF shall meet the requirements in this Section.

12.2. Special Segment

Each horizontal truss that is part of the Seismic Force Resisting System shall have a special segment that is located within the middle one-half length of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

Panels within a special segment shall either be all Vierendeel panels or all X-braced panels; neither a combination thereof nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a design strength adequate to resist a force that is at least equal to 0.25 times the nominal tensile strength of the diagonal member. Bolted connections shall not be used for web members within the special segment.

Splicing of chord members is not permitted within the special segment, nor within one-half the panel length from the ends of the special segment. Axial forces due to factored dead plus live loads in diagonal web members within the special segment shall not exceed $0.03F_yA_g$.

12.3. Nominal Strength of Special Segment Members

In the fully yielded state, the special segment shall develop vertical nominal shear strength through the nominal flexural strength of the chord members and through the nominal axial tensile and compressive strengths of the diagonal web members. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25 percent of the required vertical shear strength in the fully yielded state. The axial strength in the chord members shall not exceed 0.45 times $\phi F_y A_g$, where $\phi = 0.9$. Diagonal members in any panel of the special segment shall be made of identical sections. The end connection of diagonal web members in the special segment shall have a design strength that is at least equal to the expected nominal axial tensile strength of the web member, $R_y F_y A_g$.

12.4. Nominal Strength of Non-special Segment Members

All members and connections of STMF, except those in the special segment in Section 12.2., shall have a design strength to resist the load combination of factored gravity loads as specified in LRFD Specification Load Combinations A4-5 and A4-6 and the lateral loads necessary to develop the expected vertical nominal shear strength in all segments V_{ne} given as:

$$V_{ne} = \frac{3.75R_y M_{nc}}{L_s} + 0.075EI \frac{(L - L_s)}{L_s^3} + R_y(P_{nt} + 0.3P_{nc}) \sin \alpha \quad (12-1)$$

where

- R_y = Yield stress modification factor, see Section 6.2.
- M_{nc} = Nominal flexural strength of the chord member of the special segment, kips-in.
- EI = Flexural elastic stiffness of the chord members of the special segment, kip in.²
- L = Span length of the truss, in.
- L_s = Length of the special segment, in.
- P_{nt} = Nominal axial tension strength of diagonal members of the special segment, kips.
- P_{nc} = Nominal axial compression strength of diagonal members of the special segment, kips.
- α = angle of diagonal members with the horizontal.

12.5. Compactness

Diagonal web members within the special segment shall be made of flat bars with a width-thickness ratio that is less than or equal to 2.5. The width-thickness ratio of chord members shall not exceed the limiting λ_p values from Table I-9-1. The width-thickness ratio of angles and flanges and webs of tee sections used for chord members in the special segment shall not exceed $52/\sqrt{F_y}$.

12.6. Lateral Bracing

The top and bottom chords of the trusses shall be laterally braced at the ends of special segment, and at intervals not to exceed L_p according to LRFD Specification Section F1, along the entire length of the truss. Each lateral brace at the ends of and within the special segment shall have a design strength to resist at least 5 percent of the nominal axial compressive strength P_{nc} of the special segment chord member. Lateral braces outside of the special segment shall have a design strength to resist at least 2.5 percent of the nominal compressive strength P_{nc} of the largest adjoining chord member.

13. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

13.1. Scope

Special Concentrically Braced Frames (SCBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the Design Earthquake. SCBF have increased ductility over OCBF (see Section 14) due to lesser strength degradation when compression braces buckle. SCBF shall meet the requirements in this Section.

13.2. Bracing Members

- 13.2a. Slenderness: Bracing members shall have $Kl/r \leq 1000/\sqrt{F_y}$.
- 13.2b. Required Compressive Strength: The required strength of a bracing member in axial compression shall not exceed $\phi_c P_n$.
- 13.2c. 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 is 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 Load Combinations 4-1 or 4-2. For the purposes of this provision, a line of bracing 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.
- 13.2d. Width-thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements of braces shall meet the requirements in LRFD Specification Table B5.1 and the following requirements:
 - 1. Braces shall be compact (i.e., $\lambda < \lambda_p$). The width-thickness ratio of angles shall not exceed $52/\sqrt{F_y}$.
 - 2. Round HSS shall have an outside diameter to wall thickness ratio conforming to Table I-9-1 unless the round HSS wall is stiffened.
 - 3. Rectangular HSS shall have a flat width to wall thickness ratio conforming to Table I-9-1 unless the rectangular HSS walls are stiffened.
- 13.2e. Built-up Members: The spacing of stitches shall be such that the slenderness ratio l/r of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member.

The total design shear strength of the stitches shall be at least equal to the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

Exception: Where it can be shown that braces will buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio l/r of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

13.3. Bracing Connections

13.3.a. Required Strength: The required strength of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:

- a. The nominal axial tensile strength of the bracing member, determined as $R_y F_y A_g$.
- b. The maximum force, indicated by analysis, that can be transferred to the brace by the system.

13.3b. Tensile Strength: The design tensile strength of bracing members and their connections, based upon the limit states of tension rupture on the effective net section and block shear rupture strength, as specified in LRFD Specification Chapter D, shall be at least equal to the required strength of the brace as determined in Section 13.3a.

13.3c. Flexural Strength: In the direction that analysis indicates that the brace will buckle, the design flexural strength of the connection shall be equal to or greater than the expected nominal flexural strength $1.1 R_y M_p$ of the brace about the critical buckling axis.

Exception: Brace connections that meet the requirements in Section 13.3b., can accommodate the inelastic rotations associated with brace post-buckling deformations, and have a design strength that is at least equal to the nominal compressive strength $F_{cr} A_g$ of the brace are permitted.

13.3d. Gusset Plates: The design of gusset plates shall include consideration of buckling.

13.4. Special Bracing Configuration Special Requirements

13.4a. V-Type and Inverted-V-Type Bracing: V-type and inverted-V-type braced frames shall meet the following requirements:

1. A beam that is intersected by braces shall be continuous between columns.
2. A beam that is intersected by braces shall be designed to support the effects of all tributary dead and live loads from LRFD Specification Load Combinations A4-1, A4-2 and A4-3 assuming that the bracing is not present.

3. A beam that is intersected by braces shall be designed to resist the effects of LRFD Specification Load Combinations A4-5 and A4-6 except that a load Q_b shall be substituted for the term E . Q_b is the maximum unbalanced vertical load effect applied to the beam by the braces. This load effect shall be calculated using a minimum of P_y for the brace in tension and a maximum of 0.3 times $\phi_c P_n$ for the brace in compression.
4. The top and bottom flanges of the beam at the point of intersection of braces shall be designed to support a lateral force that is equal to 2 percent of the nominal beam flange strength $F_y b_f t_{bf}$.

Exception: Limitations 2 and 3 need not apply to penthouses, one-story buildings, nor the top story of buildings.

- 13.4b. K-Type Bracing: K-type braced frames are not permitted for SCBF.

13.5. Columns

Columns in SCBF shall meet the following requirements:

- 13.5a. Width-thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements of columns shall meet the requirements for bracing members in Section 13.2d.
- 13.5b. Splices: In addition to meeting the requirements in Section 8.3, column splices in SCBF shall be designed to develop at least the nominal shear strength of the smaller connected member and 50 percent of the nominal flexural strength of the smaller connected section. Splices shall be located in the middle one-third of the column clear height.

14. ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

14.1. Scope

Ordinary Centrically Braced Frames (OCBF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the Design Earthquake. OCBF shall meet the requirements in this Section.

14.2. Bracing Members

- 14.2a. Slenderness: Bracing members shall have $Kl/r \leq 720/\sqrt{F_y}$ except as permitted in Section 14.5.
- 14.2b. Required Compressive Strength: The required strength of a bracing member in axial compression shall not exceed 0.8 times $\phi_c P_n$.
- 14.2c. 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 is 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 Load Combinations 4-1 or 4-2. A line of bracing, for the purposes 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.

14.2d. Width-thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements in braces shall meet the requirements in LRFD Specification Table B5.1 and the following requirements:

1. Braces shall be compact or non-compact, but not slender (i.e., $\lambda < \lambda_r$). The width-thickness ratio of angles shall not exceed $52/\sqrt{F_y}$.
2. Round HSS shall have an outside diameter to wall thickness ratio conforming to Table I-9-1 unless the round HSS wall is stiffened.
3. Rectangular HSS shall have a flat width to wall thickness ratio conforming to Table I-9-1 unless the rectangular section walls are stiffened.

14.2e. Built-up Member Stitches: For all built-up braces, the first bolted or welded stitch on each side of the mid-length 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.

14.3. Bracing Connections

14.3a. Required Strength: The required strength of bracing connections (including beam-to-column connections if part of the bracing system) shall be the least of the following:

- a. The nominal axial tensile strength of the bracing member, determined as $R_y F_y A_g$.
- b. The force in the brace that results from Load Combinations 4-1 and 4-2.
- c. The maximum force, indicated by analysis, that can be transferred to the brace by the system.

14.3b. Tensile Strength: The design tensile strength of bracing members and their connections, based upon the limit states of tension rupture on the effective net section and block shear rupture strength, as specified in LRFD Specification Chapter D, shall be at least equal to the required strength of the bracing connection as determined in Section 14.3a.

14.3c. Flexural Strength: In the direction in which analysis indicates that the brace will buckle, the design flexural strength of the connection shall be equal to or greater than the expected nominal flexural strength $1.1R_y M_p$ of the brace about the critical buckling axis.

Exception: Bracing connections that meet the requirements in Section 14.3b., that can accommodate the inelastic rotations associated with brace post-buckling deformations, and that have a design strength that is at least equal to the nominal compressive strength $F_{cr} A_g$ of the brace are permitted.

14.3d. Gusset Plates: The design of gusset plates shall include consideration of buckling.

14.4. Bracing Configuration Special Requirements

14.4a. V-Type and Inverted-V-Type Bracing: V-type and inverted-V-type braced frames shall meet the following requirements:

1. The design strength of brace members shall be at least 1.5 times the required strength using LRFD Specification Load Combinations A4-5 and A4-6.
2. A beam that is intersected by braces shall be continuous between columns.
3. A beam that is intersected by braces shall be designed to support the effects of all tributary dead and live loads from LRFD Specification Load Combinations A4-1, A4-2 and A4-3 assuming that the bracing is not present.
4. The top and bottom flanges of the beam at the point of intersection of braces shall be designed to support a lateral force that is equal to 2 percent of the nominal beam flange strength $F_y b_f t_f$.

14.4b. K-Type Bracing: Buildings using K-type bracing shall not be permitted except as described in Section 14.5.

14.5. Low Buildings

When Load Combinations 4-1 and 4-2 are used to determine the required strength of the members and connections, it is permitted to design the OCBF in roof structures and buildings two stories or less in height without the special requirements of 14.2 through 14.4.

15. ECCENTRICALLY BRACED FRAMES (EBF)

15.1. Scope

Eccentrically Braced Frames (EBF) are expected to withstand significant inelastic deformations in the Links when subjected to the forces resulting from the motions of the Design Earthquake. The diagonal braces, the columns, and the beam segments outside of the Links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened Links, except where permitted in this Section. In buildings exceeding five stories in height, the upper story of an EBF system is permitted to be designed as an OCBF or an SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the Applicable Building Code. EBF shall meet the requirements in this Section.

15.2. Links

- 15.2a.** Links shall comply with the width-thickness ratios in Table I-9-1.
- 15.2b.** The specified minimum yield stress of steel used for Links shall not exceed 50 ksi.
- 15.2c.** The web of a Link shall be single thickness without doubler-plate reinforcement and without web penetrations.

- 15.2d.** Except as limited in Section 15.2f., the required shear strength of the Link V_u shall not exceed the design shear strength of the Link ϕV_n , where:

V_n = nominal shear strength of the Link, equal to the lesser of V_p or $2M_p/e$, kips.

$V_p = 0.60F_y(d - 2t_f)t_w$, kips.

$\phi = 0.9$.

e = Link length, in.

- 15.2e.** If the required axial strength P_u in a Link is equal to or less than $0.15P_y$, where P_y is equal to F_yA_g , the effect of axial force on the Link design shear strength need not be considered.

- 15.2f.** If the required axial strength P_u in a Link exceeds $0.15P_y$, the following additional requirements shall be met:

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

$$\phi = 0.9$$

$$V_{pa} = V_p \sqrt{1 - (P_u/P_y)^2} \quad (15-1)$$

$$M_{pa} = 1.18M_p[1 - (P_u/P_y)] \quad (15-2)$$

2. The length of the Link shall not exceed:

$$[1.15 - 0.5\rho(A_w/A_g)]1.6M_p/V_p \text{ when } \rho'(A_w/A_g) \geq 0.3 \quad (15-3)$$

nor

$$1.6M_p/V_p \text{ when } \rho'(A_w/A_g) < 0.3 \quad (15-4)$$

where:

$$A_w = (d_b - 2t_f)t_w$$

$$\rho' = P_u/V_u$$

- 15.2g.** 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, Δ . The Link Rotation Angle shall not exceed the following values:

- 0.08 radians for Links of length $1.6M_p/V_p$ or less.
- 0.02 radians for Links of length $2.6M_p/V_p$ or greater.
- The value determined by linear interpolation between the above values for Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

15.3. Link Stiffeners

- 15.3a.** 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 $0.75t_w$ nor $3/8$ in., whichever is larger, where b_f and t_w are the Link flange width and Link web thickness, respectively.

15.3b. 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 0.08 radians or $(52t_w - d/5)$ for Link Rotation Angles of 0.02 radians or less. Linear interpolation shall be used for values between 0.08 and 0.02 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 1.5 times b_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. Intermediate web stiffeners are not required in Links of lengths greater than $5M_p/V_p$.
5. Intermediate Link web stiffeners shall be full depth. For Links that are less than 25 in. 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 $\frac{3}{8}$ in., whichever is larger, and the width shall be not less than $(b_f/2) - t_w$. For Links that are 25 in. in depth or greater, similar intermediate stiffeners are required on both sides of the web.

15.3c. Fillet welds connecting a Link stiffener to the Link web shall have a design strength adequate to resist a force of $F_y A_{st}$, where A_{st} is the area of the stiffener. The design strength of fillet welds fastening the stiffener to the flanges shall be adequate to resist a force of $A_{st} F_y/4$.

15.4. Link-to-Column Connections

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

- 15.4a.** The Link-to-column connection design shall be based upon cyclic test results that demonstrate an inelastic rotation capability that is 20 percent greater than that calculated at the Design Story Drift, Δ . Qualifying test results shall be as described in Sections 9.2a and 9.2b., except that the inelastic rotation angle shall be as described in Section 15.2g.
- 15.4b.** Where reinforcement at the beam-to-column connection at the Link end precludes yielding of the beam over the reinforced length, the Link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such Links are used and the Link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the design strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened Link as described in Section 15.6a. Full depth stiffeners as required in Section 15.3a. shall be placed at the Link-to-reinforcement interface.

15.5. Lateral Support of Link

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

15.6. Diagonal Brace and Beam Outside of Link

15.6a. The required combined axial and flexural strength of the diagonal brace shall be the axial forces and moments generated by the expected nominal shear strength of the Link $R_y V_n$ increased by 125 percent to account for strain-hardening, where V_n is as defined in Section 15.2. The design strengths of the diagonal brace, as determined in LRFD Specification Chapter H (including Appendix H3), shall exceed the required strengths as defined above.

15.6b. The design of the beam outside the Link shall meet the following requirements:

1. The required strength of the beam outside of the Link shall be the forces generated by at least 1.1 times the expected nominal shear strength of the Link $R_y V_n$, where V_n is as defined in Section 15.2. For determining the design strength of this portion of the beam, it is permitted to multiply the design strengths determined from the LRFD Specification by R_y .
2. The beam shall be provided with lateral support where analysis indicates that support is necessary to maintain the stability of the beam. Lateral support shall be provided at both the top and bottom flanges of the beam and each shall have a required strength of at least 2 percent of the beam flange nominal strength computed as $F_y b_f t_f$.

15.6c. 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 at the end of the Link or in the Link.

15.6d. The required strength of the diagonal brace-to-beam connection at the Link end of the brace shall be at least the expected nominal strength of the brace as given in Section 15.6a. 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 an FR moment connection.

15.6e. The width-thickness ratio of the brace shall satisfy λ_p in LRFD Specification Table B5.1.

15.7. Beam-to-Column Connections

Beam-to-column connections away from Links are permitted to be designed as pinned in the plane of the web. The connection shall have a required strength to resist rotation about the longitudinal axis of the beam based upon two equal and opposite forces of at least 2 percent of the beam flange nominal strength computed as $F_y b_f t_f$ acting laterally on the beam flanges.

15.8. Required Column Strength

In addition to the requirements in Section 8, the required strength of columns shall be determined from LRFD Specification Load Combinations A4-5 and A4-6, 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 the expected nominal strength of the Link multiplied by 1.1 to account for strain-hardening. The expected nominal strength of the Link is $R_y V_n$, where V_n is as defined in Section 15.2d.

16. 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 the specifications of the Engineer of Record.

The special inspections and tests necessary to establish that the construction is in conformance with these Provisions shall be included in a quality assurance plan. The contractor's quality assurance program and qualifications, such as participation in a recognized quality certification program, shall be considered when establishing a quality assurance plan.

The minimum special inspection and testing contained in the quality assurance plan beyond that required in LRFD Specification Section M5 shall be as follows:

Visual inspection of welding shall be the primary method used to confirm that the procedures, materials and workmanship incorporated in construction are those that have been specified and approved for the project. Visual inspections shall be conducted by qualified personnel, in accordance with a written practice. Nondestructive testing of welds in conformance with AWS D1.1 shall serve as a backup, but shall not serve to replace visual inspection.

All complete-joint-penetration and partial-joint-penetration groove welded joints that are subjected to net tensile forces as part of the Seismic Force Resisting Systems in Sections 9, 10, 11, 12, 13, 14 and 15 shall be tested using approved nondestructive methods conforming to AWS D1.1.

Exception: The amount of nondestructive testing is permitted to be reduced if approved by the Engineer of Record and the regulatory agency.

Appendix S

Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections

S1. SCOPE AND PURPOSE

This Appendix includes requirements for qualifying cyclic tests of beam-to-column moment connections in Moment Frames and Link-to-column connections in Eccentrically Braced Frames, when required in these Provisions. The purpose of the testing described in this Appendix is to provide evidence that a moment connection satisfies the requirements for strength and Inelastic Rotation in these Provisions. Alternative testing requirements are permitted when approved by the Engineer of Record and the regulatory agency.

This Appendix provides only minimum recommendations for simplified test conditions. If conditions in the actual building so warrant, additional testing shall be performed to demonstrate satisfactory and reliable performance of moment connections during actual earthquake motions.

S2. SYMBOLS

The numbers in parentheses after the definition of a symbol refers to the Section number in which the symbol is first used.

- δ Deformation quantity used to control loading of Test Specimen. (S6)
- δ_y Value of deformation quantity δ at first significant yield of Test Specimen. (S6)

S3. DEFINITIONS

Prototype. The connections, member sizes, steel properties, and other design, detailing, and construction features to be used in the actual building frame.

Test Specimen. A portion of a frame used for laboratory testing, intended to model the Prototype.

Test Setup. The supporting fixtures, loading equipment, and lateral bracing used to support and load the Test Specimen.

Test Subassembly. The combination of the Test Specimen and pertinent portions of the Test Setup.

Inelastic Rotation. The permanent or plastic portion of the rotation angle between a beam and the column or between a Link and the column of the Test Specimen, measured in radians. The Inelastic Rotation shall be

computed based upon an analysis of Test Specimen deformations. Sources of Inelastic Rotation include yielding of members and connectors, yielding of connection elements, and slip between members and connection elements. Inelastic Rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of a line connecting the centerline of the inflection point of the beam or Link with the centerline of the beam at the column face.

S4. TEST SUBASSEMBLAGE REQUIREMENTS

The Test Subassembly shall replicate as closely as is practical the conditions that will occur in the Prototype during earthquake loading. The Test Subassembly shall include the following features:

1. The Test Specimen shall consist of at least a single column with beams or Links attached to one or both sides of the column.
2. Points of inflection in the test assemblage shall coincide approximately with the anticipated points of inflection in the Prototype under earthquake loading.
3. Lateral bracing of the Test Subassembly is permitted near load application or reaction points as needed to provide lateral stability of the Test Subassembly. Additional lateral bracing of the Test Subassembly is not permitted, unless it replicates lateral bracing to be used in the Prototype.

S5. ESSENTIAL TEST VARIABLES

The Test Specimen shall replicate as closely as is practical the pertinent design, detailing, construction features, and material properties of the Prototype. The following variables shall be replicated in the Test Specimen:

S5.1. Sources of Inelastic Rotation

Inelastic Rotation shall be developed in the Test Specimen by inelastic action in the same members and connection elements as anticipated in the Prototype, i.e., in the beam or Link, in the column panel-zone, in the column outside of the panel-zone, or within connection elements. The fraction of the total Inelastic Rotation in the Test Specimen that is developed in each member or connection element shall be at least 75 percent of the anticipated fraction of the total Inelastic Rotation in the Prototype that is developed in the corresponding member or connection element.

S5.2. Size of Members

1. The size of the beam or Link used in the Test Specimen shall be within the following limits:
 - a. The depth of the test beam or Link shall be no less than 90 percent of the depth of the Prototype beam or Link.
 - b. The weight per foot of the test beam or Link shall be no less than 75 percent of the weight per foot of the Prototype beam or Link.
2. The size of the column used in the Test Specimen shall properly represent the inelastic action in the column, as per the requirements in Section S5.1

Extrapolation beyond the limitations stated in this Section shall be permitted subject to qualified peer review and building official approval.

S5.3. Connection Details

The connection details used in the Test Specimen shall represent the Prototype connection details as closely as possible. The connection elements used in the Test Specimen shall be a full-scale representation of the connection elements used in the Prototype, for the member sizes being tested.

S5.4. Continuity Plates

The size and connection details of continuity plates used in the Test Specimen shall be proportioned to match the size and connection details of continuity plates used in the Prototype connection as closely as possible.

S5.5. Material Strength

The following additional requirements shall be satisfied for each member or connection element of the Test Specimen that supplies Inelastic Rotation by yielding:

1. The yield stress shall be determined by material tests on the actual materials used for the Test Specimen, as specified in Section S8. The use of yield stress values that are reported on certified mill test reports are not permitted to be used for purposes of this Section.
2. The yield stress shall not be more than 15 percent below F_{ye} for the grade of steel to be used for the corresponding elements of the Prototype. F_{ye} shall be determined in accordance with Section 6.2.

S5.6. Welds

The welds on the Test Specimen shall replicate the welds on the Prototype as closely as practicable. Additionally, welds on the Test Specimen shall satisfy the following requirements:

1. Welding shall be performed in strict conformance with a Welding Procedure Specifications (WPS) as required in AWS D1.1. The WPS essential variables shall meet the requirements in AWS D1.1 and shall be within the parameters established by the filler-metal manufacturer.
2. The specified minimum tensile strength of the filler metal used for the Test Specimen shall be the same as that to be used for the corresponding Prototype welds.
3. The specified minimum CVN toughness of the filler metal used for the Test Specimen shall not exceed the specified minimum CVN toughness of the filler metal to be used for the corresponding Prototype welds.
4. The welding positions used to make the welds on the Test Specimen shall be the same as those to be used for the Prototype welds.
5. Details of weld backing, weld tabs, access holes, and similar items used for the Test Specimen welds shall be the same as those to be used for the corresponding Prototype welds. Weld backing and weld tabs shall not be removed from the Test Specimen welds unless the corresponding weld backing and weld tabs are removed from the Prototype welds.

6. Methods of inspection and nondestructive testing and standards of acceptance used for Test Specimen welds shall be the same as those to be used for the Prototype welds.

S5.7. Bolts

The bolted portions of the Test Specimen shall replicate the bolted portions of the Prototype connection as closely as possible. Additionally, bolted portions of the Test Specimen shall satisfy the following requirements:

1. The bolt grade (e.g., ASTM A325, ASTM A490) used in the Test Specimen shall be the same as that to be used for the Prototype.
2. The type and orientation of bolt holes (standard, oversize, short slot, long slot, or other) used in the Test Specimen shall be the same as those to be used for the corresponding bolt holes in the Prototype.
3. When Inelastic Rotation is to be developed either by yielding or by slip within a bolted portion of the connection, the method used to make the bolt holes (drilling, sub-punching and reaming, or other) in the Test Specimen shall be the same as that to be used in the corresponding bolt holes in the Prototype.
4. Bolts in the Test Specimen shall have the same installation (fully tensioned or other) and faying surface preparation (no specified slip resistance, Class A slip resistance, or other) as that to be used for the corresponding bolts in the Prototype.

S6. LOADING HISTORY

S6.1. General Requirements

The Test Specimen shall be subjected to cyclic loads according to the requirements prescribed in Sections S6.2 and S6.3. Additional increments of loading beyond those prescribed in Section S6.3 are permitted.

S6.2. Test Control

The test shall be conducted by controlling the level of deformation imposed on the Test Specimen. For test control, any pertinent deformation quantity δ is permitted to be used. The value of the selected deformation quantity at first significant yield of the Test Specimen δ_y shall be determined for the purposes of test control from an analysis of the expected response of the Test Specimen.

S6.3. Loading Sequence

Loads shall be applied to the Test Specimen, up to the completion of the test, to produce the following deformations:

1. 3 cycles of loading at: $0.25\delta_y < \delta \leq 0.5\delta_y$
2. 3 cycles of loading at: $0.6\delta_y < \delta \leq 0.8\delta_y$
3. 3 cycles of loading at: $\delta = \delta_y$
4. 3 cycles of loading at: $\delta = 2\delta_y$
5. 3 cycles of loading at: $\delta = 3\delta_y$

6. 2 cycles of loading at: $\delta = 4\delta_y$
7. After completion of the loading cycles at $4\delta_y$, testing shall be continued by applying cyclic loads to produce δ equal to $5\delta_y$, $6\delta_y$, $7\delta_y$, etc. Two cycles of loading shall be applied at each incremental value of deformation.

Other loading sequences are permitted to be used to qualify the Test Specimen when they are demonstrated to be of equivalent severity.

S7. INSTRUMENTATION

Sufficient instrumentation shall be provided on the Test Specimen to permit measurement or calculation of the quantities listed in Section S9.

S8. MATERIALS TESTING REQUIREMENTS

S8.1. Tension Testing Requirements

Tension testing shall be conducted on samples of steel taken from the material adjacent to each Test Specimen. Tension-test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Section. Tension-test results shall be based upon testing that is conducted in accordance with Section S8.2. Tension testing shall be conducted and reported for the following portions of the Test Specimen:

1. Flange(s) and web(s) of beams and columns at standard locations.
2. Any element of the connection that supplies Inelastic Rotation by yielding.

S8.2. Methods of Tension Testing

Tension testing shall be conducted in accordance with ASTM A6, ASTM A370, and ASTM E8, with the following exceptions:

1. The yield stress F_y that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method at 0.002 strain.
2. The loading rate for the tension test shall replicate, as closely as practical, the loading rate to be used for the Test Specimen.

S9. TEST REPORTING REQUIREMENTS

For each Test Specimen, a written test report meeting the requirements of the regulatory agency and the requirements of this Section shall be prepared. The report shall thoroughly document all key features and results of the test. The report shall include the following information:

1. A drawing or clear description of the Test Subassembly, including key dimensions, boundary conditions at loading and reaction points, and location of lateral braces.
2. A drawing of the connection detail showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt holes, the size and grade of bolts, and all other pertinent details of the connection.
3. A listing of all other Essential Variables for the Test Specimen, as listed in Section S5.

4. A listing or plot showing the applied load or displacement history of the Test Specimen.
5. A plot of the applied load versus the displacement of the Test Specimen. The displacement reported in this plot shall be measured at or near the point of load application. The locations on the Test Specimen where the loads and displacements were measured shall be clearly indicated.
6. A plot of beam moment versus total Inelastic Rotation. The beam moment and the total Inelastic Rotation shall be computed with respect to the face of the column.
7. The total Inelastic Rotation developed by the Test Specimen. The components of the Test Specimen contributing to the total Inelastic Rotation due to yielding or slip shall be identified. The portion of the total Inelastic Rotation contributed by each component of the Test Specimen shall be reported. The method used to compute Inelastic Rotations shall be clearly shown.
8. A chronologic listing of significant test observations, including observations of yielding, slip, instability, and fracture of any portion of the Test Specimen as applicable.
9. The controlling failure mode for the Test Specimen. If the test is terminated prior to failure, the reason for terminating the test shall be clearly indicated.
10. The results of the material tests specified in Section S8.
11. The Welding Procedure Specifications (WPS) and welding inspection reports.

Additional drawings, data, and discussion of the Test Specimen or test results are permitted to be included in the report.

S10. ACCEPTANCE CRITERIA

For each connection used in the actual frame, at least two tests are required for each condition in which the Essential Variables, as listed in Section S4, remain within the required limits. Both tests shall satisfy the criteria stipulated in Sections 8.5, 9.2, 10.2, or 15.4, as applicable. In order to satisfy Inelastic Rotation requirements, each Test Specimen shall sustain the required rotation for at least one complete loading cycle.

Part II

Composite Structural Steel and Reinforced Concrete Buildings

Part II Glossary

Applicable Building Code. The building code under which the building is designed.

Boundary Members. Portion along wall and diaphragm edges strengthened with structural steel sections and/or longitudinal steel reinforcement and transverse reinforcement.

Collector Elements. Member that serves to transfer forces between floor diaphragms and the members of the Seismic Force Resisting System.

Composite Beam. A structural steel beam that is either an unencased steel beam that acts integrally with a concrete or composite slab using shear connectors or a fully reinforced-concrete-encased steel beam.

Composite Brace. A reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section that is used as a brace.

Composite Column. A reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section that is used as a column.

Composite Plate–Concrete Shear Wall. A wall that consists of a steel plate with reinforced concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel plate.

Composite Shear Wall. A reinforced concrete wall that has unencased or reinforced-concrete-encased structural steel sections as Boundary Members.

Composite Slab. A concrete slab that is supported on and bonded to a formed steel deck and that acts as a diaphragm to transfer force to and between elements of the Seismic Force Resisting System.

Concrete-filled Composite Column. Round or rectangular structural steel section that is filled with concrete.

Coupling Beam. A structural steel or composite beam that connects adjacent reinforced concrete wall elements so that they act together to resist lateral forces.

Design Strength. The design resistance (force, moment, stress, as appropriate) that is provided by an element or connection; the product of the nominal strength and the resistance factor.

Encased Composite Beam. A structural steel beam that is completely encased in reinforced concrete that is cast integrally with the slab and for which full composite action is provided by bond between the structural steel and reinforced concrete.

Encased Composite Column. A structural steel column (rolled or built-up) that is completely encased in reinforced concrete.

Face Bearing Plates. Stiffeners that are attached to structural steel beams that are embedded in reinforced concrete walls or columns. The plates are located at the face

of the reinforced concrete to provide confinement and to transfer forces to the concrete through direct bearing.

Fully Composite Beam. A composite beam that has a sufficient number of shear connectors to develop the nominal plastic flexural strength of the composite section.

Load-Carrying Reinforcement. Reinforcement in composite members that is designed and detailed to resist the required loads.

Nominal Strength. The strength of a member or cross-section to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas that are 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.

Partially Composite Beam. An unencased composite beam with a nominal flexural strength that is controlled by the strength of the shear stud connectors.

Partially Restrained Composite Connection. Partially restrained connections as defined in the LRFD Specification that connect partially or fully composite beams to steel columns with flexural resistance provided by a force couple achieved with steel reinforcement in the slab and a steel seat angle or similar connection at the bottom flange.

Reinforced-Concrete-Encased Shapes. Structural steel sections that are encased in reinforced concrete.

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

Restraining Bars. Steel reinforcement in composite members that is not designed to carry required forces, but is provided to facilitate the erection of other steel reinforcement and to provide anchorage for stirrups or ties. Generally, such reinforcement is not spliced to be continuous.

Static Yield Strength. The strength of a structural member or connection that is determined on the basis of testing that is conducted under slow monotonic loading until failure.

1. SCOPE

These Provisions are intended for the design and construction of composite structural steel and reinforced concrete members and connections in the Seismic Force Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response.

These Provisions shall be applied in conjunction with the AISC *Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings*, hereinafter referred to as the LRFD Specification. All members and connections in the Seismic Force Resisting System shall have a design strength as provided in the LRFD Specification to resist load combinations A4-1 through A4-6 and shall meet the requirements in these Provisions. The applicable requirements in Part I shall be used for the design of structural steel components in composite systems. Reinforced-concrete members subjected to seismic forces shall meet the requirements in ACI 318, except as modified in these provisions. When the design is based upon elastic analysis, the stiffness properties of the component members of composite systems shall reflect their condition at the onset of significant yielding of the building.

Part II includes a Glossary, which is specifically applicable to this Part. The Part I Glossary is also applicable to Part II.

2. REFERENCED CODES AND STANDARDS

The documents referenced in these provisions shall include those listed in Part I Section 2 with the following additions and modifications:

American Concrete Institute
ACI 318-95¹

American Iron and Steel Institute
Specification for the Design of Cold-Formed Steel Structural Members, 1996 Edition²

American Society of Civil Engineers
ASCE 3-91

3. SEISMIC DESIGN CATEGORIES

Seismic provisions, the required strength for each Seismic Design Category, Seismic Use Group or Seismic Zone and the limitations for height and irregularities shall be as specified in the Applicable Building Code; or, when no code is applicable, as dictated by the conditions involved.

¹The alternative load and strength reduction (resistance) factors specified in ACI 318 Appendix C shall be used, except that the load factor on E shall be revised to be consistent with that specified in the Applicable Building Code.

²The LRFD portions of this document, which provides an integral treatment of LRFD and ASD, shall be used.