ANSI/AISC 341-02
An American National Standard

Seismic Provisions for Structural Steel Buildings

Supersedes the *Seismic Provisions for Structural Steel Buildings* dated April 15, 1997 including Supplements No. 1 and 2 and all previous versions

Approved by the
AISC Committee on Specifications and
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DEDICATION

Professor Egor Popov

This edition of the AISC Seismic Provisions is dedicated to the memory of Professor Egor Popov. Professor Popov was a Professor for over 50 years at the University of California at Berkeley, and a long time member of the AISC Committee on Specifications. Professor Popov focused a major portion of his career improving the understanding and seismic performance of steel structures. He was instrumental in the development of seismic design provisions for steel structures for over thirty years, and initiated the activity of AISC in this regard in the late 1980's. As Chair of TC113 (the predecessor of TC9), he led the publication of the first two editions of the AISC Seismic Provisions. Until the time of his death at the age of 88 early in 2001, Professor Popov remained a very active member of TC9 in the role of Vice Chair. His contributions to the development of these provisions and understanding of the seismic performance of steel buildings is unequaled, and will long be remembered and appreciated by AISC, the steel industry and the structural engineering profession. It is entirely fitting that these provisions be dedicated to the memory of Professor Egor Popov.

PREFACE

(This Preface is not a part of ANSI/AISC 341-02, *Seismic Provisions for Structural Steel Buildings,* but is included for information purposes only.)

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 for it to also cover the many special and unique problems encountered within the full range of structural design practice. This document, the AISC *Seismic Provisions for Structural Steel Buildings* (hereafter referred to as Seismic Provisions) is a separate consensus document that addresses one such topic: the design and construction of structural steel and composite structural steel/reinforced concrete building systems for seismic demands.

These Provisions are presented in three parts: Part I is intended for the design and construction of structural steel buildings, using LRFD; 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. In addition, three appendices, 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 previous edition of the AISC Seismic Provisions for Structural Steel Buildings, published on April 15, 1997, incorporated many of the early advances achieved as part of the FEMA/SAC program and other investigations and developments related to the seismic design of steel buildings. Recognizing that rapid and significant changes in the knowledge base were occurring for the seismic design of steel buildings, especially Moment Frames, the AISC Specifications Committee committed to generating frequent supplements to the Seismic Provisions. This commitment was intended to keep the provisions as current as possible. The first such supplement was completed and published on February 15, 1999, Supplement No. 1 to the 1997 AISC Seismic Provisions. Supplement Number 2 to the 1997 AISC Seismic Provisions was published on November 10, 2000.

This edition of the AISC Seismic Provisions incorporates Supplements No. 1 (February 15, 1999) and No. 2 (November 10, 2000) to the 1997 Seismic Provisions. This version also includes Errata to Sections 8.4 and

9.9. Additional revisions resulted from considering new information generated by the FEMA/SAC project, which culminated late in 2000, and other sources. These provisions were also modified to be consistent with the ASCE 7-02 document, *Minimum Design Loads for Buildings and Other Structures.* This allows these provisions to be incorporated by reference into both the 2003 IBC and 2002 NFPA 5000 building codes that use ASCE 7-02 as their basis for design loadings. Because the scope of changes that have been made to these provisions since 1997 is so large, they are being republished in their entirety. A major update to the commentary to these provisions is also provided. Specific changes to these provisions include the following:

- A clarification to the glossary to verify that chord and collector/drag elements in floor diaphragms are considered to be part of the Seismic Load Resisting System.
- Additional requirements for the toughness of filler metals to be used in complete-joint-penetration groove welds in intermediate and Special Moment Frame systems.
- A revision to clarify member slenderness ratio requirements and better coordinate with the LRFD provisions.
- Increasing the Moment Frame column splice requirements to reflect the FEMA/SAC recommendations.
- Requiring that splices of columns that are not part of the Moment Frames develop a minimum shear force.
- Clarifying Column Base design demands for various systems.
- Adding a section on the use of H-pile members.
- Clarifying lateral bracing requirements of Moment Frame beams, including the provision of a required stiffness to be consistent with Section 3 of LRFD.
- Increasing SMF web Connection design requirements to be consistent with the FEMA/SAC recommendations.
- Adding a new appendix (Appendix P) that defines procedures to be used in the pre-qualification of moment Connections.
- Incorporating FEMA/SAC recommendations for weld access holes in OMF systems.
- Incorporating FEMA/SAC recommendations for the removal of weld backing and run-off tabs in OMF systems, including grinding surfaces to adequate smoothness.
- Dual units format. Values and equations are given in both U.S. customary and metric units. The metric conversions (given in parentheses following the U.S. units) are based on IEEE/ASTM SI 10, *Standard for Use of the International System of Units (SI): The Modern Metric System.* The equations are non-dimensionalized where possible by factoring out material constants, such as E.

The AISC Committee on Specifications, Task Committee 9—Seismic

Provisions is responsible for the ongoing development of these Provisions. The AISC Committee on Specifications gives final approval of the document through an ANSI accredited balloting process, and has enhanced these Provisions through careful scrutiny, discussion, and suggestions for improvement. AISC further acknowledges the significant contributions of several groups to the completion of this document: the Building Seismic Safety Council (BSSC), the SAC Joint Venture, the Federal Emergency Management Agency (FEMA), the National Science Foundation (NSF), and the Structural Engineers Association of California (SEAOC).

The reader is cautioned that professional judgment must be exercised when data or recommendations in these provisions are applied, as described more fully in the disclaimer notice preceding the Preface.

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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} (mm^2) (I-8)
A_g	Gross area, in.^{2} (mm^2) (I-9)
A_s	Cross-sectional area of structural steel elements in composite members, in.^{2} (mm^2) (II-6)
A_{sh}	Minimum area of tie reinforcement, in.^{2} (mm^2) (II-6)
A_{sp}	Horizontal area of the steel plate in composite shear wall, in.^{2} (mm^2) (II-17)
A_{st}	Area of Link stiffener, in.^{2} (mm^2) (I-15)
A_w	Link web area, in.^{2} (mm^2) (I-15)
D	Dead load due to the weight of the structural elements and permanent features on the building, kips (N) (I-9)
	Outside diameter of round HSS, in. (mm) (Table I-8-1)
E	Effect of horizontal and vertical earthquake-induced loads (Glossary)
E_s	Modulus of elasticity of steel, E_s = 29,000 ksi (200 000 MPa) (I-8, II-6)
E_sI	Flexural elastic stiffness of the chord members of the special segment, kip-in.^{2} (N-mm^2) (I-12)
F_y	Specified minimum yield stress of the type of steel to be used, ksi (MPa). 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 a yield point). (I-6)
F_{yb}	F_y of a beam, ksi (MPa) (I-9)
F_{yc}	F_y of a column, ksi (MPa) (I-9)
F_{yf}	F_y of column flange, ksi (MPa) (I-8)
F_{yh}	Specified minimum yield strength of transverse reinforcement, ksi (MPa) (II-6)
F_{yw}	F_y of the Panel Zone steel, ksi (MPa)
F_u	Specified minimum tensile strength, ksi (MPa) (I-7)
H	Height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, in. (mm) (I-8)
K	Effective length factor for prismatic member (I-13)
L	Live load due to occupancy and moveable equipment, kips (kN) (I-9)
	Span length of the truss, in. (mm) (I-12)
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, in. (mm) (I-12)
L_s	Length of the special segment, in. (mm) (I-12)
M_n	Nominal flexural strength, kip-in. (N-mm) (I-11)
M_{nc}	Nominal flexural strength of the chord member of the special segment, kip-in. (N-mm) (I-12)
M_p	Nominal plastic flexural strength, kip-in. (N-mm) (I-9)
M_{pa}	Nominal plastic flexural strength modified by axial load, kip-in. (N-mm) (I-15)
M_{pc}	Nominal plastic flexural strength of the column, kip-in. (N-mm) (I-8)
M_v	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline, kip-in. (N-mm) (I-9)
M_u	Required flexural strength of a member or Joint, kip-in. (N-mm) (I-9)

P_n	Nominal axial strength of a column, kips (N) (I-8) Nominal axial strength of a Composite Column, kips (N) (II-6)
P_{nc}	Nominal axial compressive strength of diagonal members of the special segment, kips (N) (I-12)
P_{nt}	Nominal axial tensile strength of diagonal members of the special segment, kips (N) (I-12)
P_o	Nominal axial strength of a Composite Column at zero eccentricity, kips (N) (II-6)
P_u	Required axial strength of a column or a Link, kips (N) (I-8) Required axial strength of a Composite Column, kips (N) (II-9)
P_{uc}	Required axial strength of a column in compression, kips (N) (I-9)
P_y	Nominal axial yield strength of a member, which is equal to F_yA_g, kips (N) (I-9)
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces, kips (N) (I-13)
R_n	Nominal Strength
R_u	Required strength (I-9)
R_v	Panel Zone nominal shear strength (I-9)
R_y	Ratio of the Expected Yield Strength to the minimum specified yield strength F_y (I-6)
S	Snow load, kips (N) (I-9)
V_n	Nominal shear strength of a member, kips (N) (I-15)
V_{ns}	Nominal shear strength of the steel plate in a composite plate shear walls, kips (N) (II-17)
V_p	Nominal shear strength of an active Link, kips (N) (I-15)
V_{pa}	Nominal shear strength of an active Link modified by the axial load magnitude, kips (N) (I-15)
V_u	Required shear strength of a member, kips (N) (I-9)
Y_{con}	Distance from top of steel beam to top of concrete slab or encasement, in. (mm) (II-6)
Z	Plastic section modulus of a member, in.^{3} (mm^3) (I-9)
Z_b	Plastic section modulus of the beam, in.^{3} (mm^3) (I-9)
Z_c	Plastic section modulus of the column, in.^{3} (mm^3) (I-9)
a	Angle that diagonal members make with the horizontal (I-12)
b	Width of compression element as defined in LRFD Specification Section B5.1, in. (mm) (Table I-8-1)
b_{cf}	Width of column flange, in. (mm) (I-9)
b_f	Flange width, in. (mm) (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. (mm) (II-6)
d	Nominal fastener diameter, in. (mm) (I-7)
d_b	Overall beam depth, in. (mm) (I-9)
d_c	Overall column depth, in. (mm) (I-9)
d_z	Overall Panel Zone depth between Continuity Plates, in. (mm) (I-9)
e	EBF Link length, in. (mm) (I-15)
f'_c	Specified compressive strength of concrete, ksi (MPa) (II-6)
h_{cc}	Cross-sectional dimension of the confined core region in Composite Columns measured center-to-center of the transverse reinforcement, in. (mm) (II-6)
l	Unbraced length between stitches of built-up bracing members, in. (mm) (I-13) Unbraced length of compression or bracing member, in. (mm) (I-13)
r	Governing radius of gyration, in. (mm) (I-9)
r_y	Radius of gyration about y axis, in. (mm) (I-9)

s — Spacing of transverse reinforcement measured along the longitudinal axis of the structural composite member, in. (mm) (II-6)

t — Thickness of connected part, in. (mm) (I-7)
Thickness of element, in. (mm) (Table I-8-1)
Thickness of column web or doubler plate, in. (mm) (I-9)

t_{bf} — Thickness of beam flange, in. (mm) (I-9)

t_{cf} — Thickness of column flange, in. (mm) (I-9)

t_f — Thickness of flange, in. (mm) (I-9)

t_p — Thickness of Panel Zone including doubler plates, in. (mm) (I-9)

t_w — Thickness of web, in. (mm) (Table I-8-1)

w_z — Width of Panel Zone between column flanges, in. (mm) (I-9)

z_b — Minimum plastic section modulus at the Reduced Beam Section, in.^{3} (mm^3) (I-9)

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_{ps} — Limiting slenderness parameter for compact element (Table I-8-1)

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)
Resistance Factor for the shear strength of a Composite Column (II-6)

γ — Link Rotation Angle (S2)

PART I. STRUCTURAL STEEL BUILDINGS

Glossary

The first letter(s) of words or terms that appear in this glossary are generally capitalized throughout these Provisions.

Applicable Building Code. The building code under which the building is designed. In the absence of an Applicable Building Code, the loads and load combinations shall be those stipulated in ASCE 7.

Amplified Seismic Load. The horizontal component of earthquake load E multiplied by Ω_o, where E and the horizontal component of E are defined in the Applicable Building Code.

Authority Having Jurisdiction. The organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this standard.

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.

Column Base. The assemblage of plates, connectors, bolts, and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

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 (drift under the Design Earthquake, including the effects of inelastic action), 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 Dual System is 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 probable yield strength of the material, equal to the minimum specified yield strength, F_y, multiplied by R_y .

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

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

Interstory Drift Angle. Interstory displacement divided by story height, radians.

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-Area. An area of potentially reduced notch-toughness located in the web-to-flange fillet area. See Figure C-I-6.1.

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 Bracing 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 inelastic angle between the Link and the beam outside of the Link when the total story drift is equal to the Design Story Drift.

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). A connection with insufficient rigidity to maintain the angles between connected members in original alignment after load is applied.

Prequalified Connections. Connections that comply with the requirements of Appendix P.

Reduced Beam Section. A 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 Load Resisting System. The assembly of structural elements in the building that resists seismic loads, including struts, collectors, chords, diaphragms and trusses.

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 13 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 12.

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.

Zipper Column. A vertical (or nearly vertical) strut connecting the brace-to-beam intersection of an Inverted-V-Braced Frame at one level to the brace-to-beam intersection at another level. See Figure C-I-13.3(b).

1. SCOPE

These Provisions are intended for the design and construction of structural steel members and connections in the Seismic Load 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 Specification for Structural Steel Buildings*, hereinafter referred to as the LRFD Specification. All members and connections in the Seismic Load Resisting System shall have a Design Strength as required in the LRFD Specification, and shall also meet all of the additional requirements in these Provisions.

Part I includes a Glossary, which is specifically applicable to this Part, and Appendices P, S, and X.

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 Concrete Institute (ACI)
Building Code Requirements for Structural Concrete, ACI 318-02

American Institute of Steel Construction (AISC)
Load and Resistance Factor Design Specification for Structural Steel Buildings, December 27, 1999
Load and Resistance Factor Design Specification for the Design of Steel Hollow Structural Sections, November 10, 2000
Load and Resistance Factor Design Specification for Single Angle Members, November 10, 2000

American Society of Civil Engineers (ASCE)
Minimum Design Loads for Buildings and Other Structures, ASCE 7-02

American Society for Testing and Materials (ASTM)
Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling, ASTM A6/A6M-01
Standard Specification for Carbon Structural Steel, ASTM A36/A36M-00
Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless, ASTM A53/A53M-01
Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates, ASTM A283/A283M-00
Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength, ASTM A325-01

Standard Specification for High-Strength Bolts for Structural Steel Joints [Metric], ASTM A325M-00
Standard Test Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370-02e1
Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength, ASTM A490-00
Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric], ASTM A490M-00
Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500-01
Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing, ASTM A501-01
Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality, ASTM A529/A529M-00
Standard Specification for High-Strength Low-Allow Columbium-Vanadium Structural Steel, ASTM A572/A572M-00a
Standard Specification for High-Strength Low-Allow Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4 in. [100 mm] Thick, ASTM A588/A588M-00a
Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing, ASTM A618-01
Standard Specification for Sampling Procedure for Impact Testing of Structural Steel, ASTM A673/A673M-95
Standard Specification for Cold-formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance, ASTM A847-99a
Standard Specification for High-Strength Low-Allow Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST), ASTM A913/A913M-00a
Standard Specification for Steel for Structural Shapes for Use in Building Framing, ASTM A992/A992M-00
Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength, ASTM F1852-00

American Welding Society
Structural Welding Code – Steel, AWS D1.1:2002

Research Council on Structural Connections
Specification for Structural Joints Using ASTM A325 or A490 Bolts, June 23, 2000

3. GENERAL SEISMIC DESIGN REQUIREMENTS

The Required Strength and other seismic provisions for Seismic Design Categories (SDCs), Seismic Use Groups or Seismic Zones and the limitations on height and irregularity shall be as specified in the Applicable Building Code.

4. LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS

4.1. Loads and Load Combinations

The loads and load combinations shall be as stipulated by the Applicable Building Code (see Glossary). Where Amplified Seismic Loads are required by these provisions, the horizontal earthquake load *E* (as defined in the Applicable Building Code) shall be multiplied by the overstrength factor Ω_o prescribed by the Applicable Building Code. In the absence of a specific definition of Ω_o, the value for Ω_o shall be as listed in Table I-4-1.

TABLE I-4-1
System Overstrength Factor, Ω_o

Seismic Load Resisting System	Ω_o
All moment-frame systems meeting Part I requirements	3
Eccentrically Braced Frames (EBF) meeting Part I requirements	2½
All other systems meeting Part I requirements	2

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 Load 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 Load Resisting Systems described in Sections 9, 10, 11, 12, 13, 14 and 15 shall meet one of the following ASTM Specifications: A36/A36M, A53/A53M, A500 (Grade B or C), A501, A529/A529M, A572/A572M (Grade 42 (290), 50 (345) or 55 (380)), A588/A588M, A913/A913M (Grade 50 (345) or 65 (450)), or A992/A992M. The steel used for Column Base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D. The specified minimum yield strength of steel to be used for members in which inelastic behavior is expected shall not exceed 50 ksi (345 MPa) 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.

No thermal treatment of weldment or test specimens is permitted, except that machined tensile test specimens may be aged at 200°F (93°C) to 220°F (104°C) for up to 48 hours, then cooled to room temperature before testing.

6.2. Material Properties for Determination of Required Strength

When required in these Provisions, the Required Strength of a connection or member shall be determined from the Expected Yield Strength R_yF_y, of the connected member, where 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 as given in Table I-6-1. Other values of R_y are permitted to be used if the value of the Expected Yield Strength is determined by testing that is conducted in accordance with the requirements for the specified grade of steel.

When both the Required Strength and the Design Strength calculations are made for the same member or connecting element, it is permitted to apply R_y to F_y in the determination of the Design Strength.

6.3. Notch-toughness Requirements

When used as members in the Seismic Load Resisting System, ASTM A6/A6M Groups 3, 4, and 5 shapes with flanges 1½ in. (38 mm) thick and thicker, and plates that are 2-in. (50 mm) thick or thicker shall have a minimum Charpy V-Notch (CVN) toughness of 20 ft-lbs (27 J) at 70°F (21°C), 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 Load Resisting System shall meet the requirements in LRFD Specification Chapter J, except as modified in this Section.

7.2. Bolted Joints

All bolts shall be pretensioned 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.

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

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.

TABLE I-6-1
R_y Values for Different Member Types

Application	R_y
Hot-rolled structural shapes and bars	
ASTM A36/A36M	1.5
ASTM A572/A572M Grade 42 (290)	1.3
ASTM A992/A992M	1.1
All other grades	1.1
Hollow Structural Sections	
ASTM A500, A501, A618 and A847	1.3
Steel Pipe	
ASTM A53/A53M	1.4
Plates	1.1
All other products	1.1

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 $2.4dtF_u$.

Bolted connections for members that are a part of the Seismic Load 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

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.

7.3a. General Requirements

All welds used in members and connections in the Seismic Load Resisting System shall be made with a filler metal that can produce welds that have a minimum Charpy V-Notch toughness of 20 ft-lbs (27 J) at minus 20°F (minus 29°C), as determined by AWS classification or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these Provisions.

7.3b. Additional Requirements in Special Moment Frames and Intermediate Moment Frames

For structures in which the steel frame is normally enclosed and maintained at a temperature of 50°F (10°C) or higher, the following CJP welds in special and Intermediate Moment Frames shall be made with filler metal capable of providing a minimum Charpy V-Notch toughness of 20 ft-lbs (27 J) at minus 20°F (minus 29°C) as determined by AWS classification test methods and 40 ft-lbs (54 J) at 70°F (21°C) as determined by Appendix X or other approved method:

(1) Welds of beam flanges to columns

(2) Groove welds of shear tabs and beam webs to columns

(3) Column splices

For structures with service temperatures lower than 50°F (10°C), these qualification temperatures shall be reduced accordingly.

7.3c. For members and connections that are part of the Seismic Load Resisting System, discontinuities located within a plastic hinging zone defined below, 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.

7.4. Other Connections

Welded shear studs shall not be placed on beam flanges within the zones of expected plastic hinging. The length of a plastic hinging zone shall be defined as one-half of the depth of the beam on either side of the theoretical hinge point. Decking arc-spot welds as required to secure decking shall be permitted. Decking attachments that penetrate the beam flanges shall not be used in the plastic hinging zone.

Welded, bolted, screwed, or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping, or other construction shall not be placed within the expected zone of plastic deformations of members of the Seismic Load Resisting System. Outside the expected zone of plastic deformation area, calculations, based on the expected moment, shall be made to demonstrate the adequacy of the member net section when connectors that penetrate the member are used.

Exception: Welded shear studs and other connections are permitted where they have been included in the connection tests used to qualify the connection.

8. MEMBERS

8.1. Scope

Members in the Seismic Load Resisting System shall meet the requirements in the LRFD Specification and those of this Section. For members that are not part of the Seismic Load Resisting System, see Section 8.4c.

8.2. Local Buckling

Where required by these Provisions, members of the Seismic Load Resisting System shall meet the λ_p limitation in Table B5.1 in the LRFD Specification and the λ_{ps} limitations of Table I-8-1.

TABLE I-8-1

Limiting Width Thickness Ratios λ_{ps} for Compression Elements

	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios λ_{ps} (seismically compact)
Unstiffened Elements	Flanges of I-shaped rolled, hybrid or welded beams [a], [b], [f], [h]	b/t	$0.30\sqrt{E_s / F_y}$
	Flanges of I-shaped rolled, hybrid or welded columns [a], [c]	b/t	$0.30\sqrt{E_s / F_y}$
	Flanges of channels, angles and I-shaped rolled, hybrid or welded beams and braces [a], [d], [h]	b/t	$0.30\sqrt{E_s / F_y}$
	Flanges of I-shaped rolled, hybrid or welded columns [a], [e]	b/t	$0.38\sqrt{E_s / F_y}$
	Flanges of H-pile sections	b/t	$0.45\sqrt{E_s / F_y}$
	Flat bars[g]	b/t	2.5
	Legs of single angle, legs of double angle members with separators, or flanges of tees [h]	b/t	$0.30\sqrt{E_s / F_y}$
	Webs of tees [h]	d/t	$0.30\sqrt{E_s / F_y}$

8.3. Column Strength

When $P_u/\phi P_n$ is greater than 0.4 without consideration of the Amplified Seismic Load, the following requirements shall be met:

(1) The required axial compressive and tensile strength, considered in the absence of any applied moment, shall be determined using the load combinations stipulated by the Applicable Building Code including the Amplified Seismic Load.

(2) The Required Strengths 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.

TABLE I-8-1 (cont.)
Limiting Width Thickness Ratios λ_{ps} for Compression Elements

	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios λ_{ps} (seismically compact)
Stiffened Elements	Webs in flexural compression in beams in SMF, Section 9, unless noted otherwise [a]	h/t_w	$2.45\sqrt{E_s/F_y}$
	Other webs in flexural compression [a]	h/t_w	$3.14\sqrt{E_s/F_y}$
	Webs in combined flexure and axial compression [a], [b], [c], [d], [e], [f], [h]	h/t_w	for $P_u/\phi_b P_y \le 0.125$ $3.14\sqrt{\frac{E_s}{F_y}}\left(1-1.54\frac{P_u}{\phi_b P_y}\right)$ for $P_u/\phi_b P_y > 0.125$ $1.12\sqrt{\frac{E_s}{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right)$
	Round HSS in axial and/or flexural compression [d], [h]	D/t	$0.044\ E_s/F_y$
	Rectangular HSS in axial and/or flexural compression [d], [h]	b/t or h/t_w	$0.64\sqrt{E_s/F_y}$
	Webs of H-Pile sections	h/t_w	$0.94\sqrt{E_s/F_y}$

[a] For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.
[b] Required for beams in SMF, Section 9.
[c] Required for columns in SMF, Section 9, unless the ratios from Equation 9-3 are greater than 2.0 where it is permitted to use λ_p in LRFD Specification Table B5.1.
[d] Required for beams and braces in SCBF, Section 13.
[e] It is permitted to use λ_p in LRFD Specification Table B5.1 for columns in STMF, Section 12 and EBF, Section 15.
[f] Required for Link in EBF, Section 15.
[g] Diagonal web members within the special segment of STMF, Section 12.
[h] Chord members of STMF, Section 12.

8.4. Column Splices

8.4a. General

The Required Strength of column splices shall equal the Required Strength of the columns, including that determined from Section 8.3.

The centerline of column splices made with fillet welds or partial-joint-penetration groove welds shall be located 4 ft. (1.2 m) or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 8 ft. (2.4 m), splices shall be at half the clear height.

Welded column splices that are subject to a calculated net tensile stress determined using the load combinations stipulated by the Applicable Building Code including the

Amplified Seismic Load, shall be made using filler metal with Charpy V-Notch toughness as required in Section 7.3a 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 Design Strength for each flange shall be at least 0.5 times $R_yF_yA_f$, where R_yF_y is the Expected Yield Strength of the column material and A_f is the flange area of the smaller column connected.

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.

8.4b. Column Web Splices

Column web splices shall be either bolted or welded, or welded to one column and bolted to the other. In Moment Frames using bolted splices to develop the Required Strength, plates or channels shall be used on both sides of the column web.

8.4c. Columns Not Part of the Seismic Load Resisting System

In moment frame buildings, splices of columns that are not a part of the Seismic Load Resisting System shall satisfy the following:

(1) They shall be located 4 ft. (1.2 m) or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 8 ft. (2.4 m), splices shall be at half the clear height.

(2) The column splices shall have sufficient design shear strength with respect to both orthogonal axes of the column to resist a shear force equal to M_{pc}/H, where M_{pc} is the nominal plastic flexural strength of the column for the direction in question, and H is the story height.

8.5. Column Bases

The connection of the structure frame elements to the Column Base and the connection of the Column Base to the foundations shall be adequate to transmit the forces for which the frame elements were required to be designed. Design of concrete elements at the Column Base, including anchor rod embedment and reinforcement steel, shall be in accordance with ACI 318. The seismic loads to be transferred to the foundation soil interface shall be as required by the Applicable Building Code.

8.6. H-Piles

8.6a. Design of H-Piles

Design of H-piles shall comply with the provisions of the AISC LRFD Specification regarding design of members subjected to combined loads. The width-thickness ratios of member elements shall meet the λ_{ps} limitations of Table I-8-1.

8.6b. Batter H-Piles

If batter (sloped) and vertical piles are used in a pile group, the vertical piles shall be designed to support combined effects of the dead and live loads without the participation of batter piles.

8.6c. Tension in H-Piles

Tension in the pile shall be transferred to the pile cap by mechanical means such as shear keys, rebars or studs welded to the embedded portion of pile. A length of pile below the bottom of the pile cap equal to at least the overall depth of the pile cross section shall be free of attachments and welds.

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. Requirements

All beam-to-column joints and connections used in the Seismic Load Resisting System shall satisfy the following three requirements:

(1) The connection must be capable of sustaining an Interstory Drift Angle of at least 0.04 radians.

(2) The required flexural strength of the connection, determined at the column face, must equal at least 80 percent of the nominal plastic moment of the connected beam at an Interstory Drift Angle of 0.04 radians.

(3) The required shear strength V_u of the connection shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from the application of a moment of $2[1.1R_yF_yZ/$distance between plastic hinge locations]. Alternatively, a lesser value of V_u is permitted if justified by analysis.

Connections that accommodate the required Interstory Drift Angle within the connection elements and provide the required flexural and shear strengths noted above are permitted, provided it can be demonstrated by analysis that the additional drift due to connection deformation can be accommodated by the building. Such analysis shall include effects of overall frame stability including second order effects.

9.2b. Conformance Demonstration

All beam-to-column joints and connections used in the Seismic Load Resisting System shall be demonstrated to satisfy the requirements of Section 9.2a by one of the following:

(a) Use a connection Prequalified for SMF in accordance with Appendix P.

(b) Provide qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in research literature or documented tests performed for other projects that are demonstrated to represent project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

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

9.3a. Shear Strength

The required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested connection. As a minimum, the required shear strength R_u of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength $\phi_v R_v$ of the panel zone shall be determined using $\phi_v = 1.0$.

(a) When $P_u \leq 0.75P_y$,

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

where

t_p = total thickness of Panel Zone including doubler plate(s), in. (mm)
d_c = overall column depth, in. (mm)
b_{cf} = width of the column flange, in. (mm)
t_{cf} = thickness of the column flange, in. (mm)

d_b = overall beam depth, in. (mm)
F_y = specified minimum yield strength of the Panel Zone steel, ksi (MPa)

(b) When $P_u > 0.75P_y$, R_v shall be calculated using LRFD Specification Equation K1-12.

9.3b. Panel Zone Thickness

The individual thicknesses t of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90 \tag{9-2}$$

where

t = thickness of column web or doubler plate, in. (mm)
d_z = Panel Zone depth between Continuity Plates, in. (mm)
w_z = Panel Zone width between column flanges, in. (mm)

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

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. Where employed, the Reduced Beam Section shall meet the required strength as specified in Section 9.2a(2).

Beams and columns shall satisfy the width-thickness limitations given in Table I-8-1.

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:

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0 \tag{9-3}$$

where

ΣM^*_{pc} = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. ΣM^*_{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 $\Sigma M^*_{pc} = \Sigma Z_c(F_{yc}-P_{uc}/A_g)$. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

ΣM^*_{pb} = the sum of the moment(s) in the beam(s) at the intersection of the beam and column centerlines. ΣM^*_{pb} is determined by summing the projections of the expected beam flexural strength(s) at the plastic hinge location(s) to the column centerline. It is permitted to take $\Sigma M^*_{pb} = \Sigma(1.1R_yF_{yb}Z_b+M_v)$, 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 ΣM^*_{pb} from test results as required in Section 9.2b or by analysis based upon the tests. When connections with Reduced Beam Sections are used, it is permitted to take $\Sigma M^*_{pb} = \Sigma(1.1R_yF_{yb}z_b+M_v)$.

A_g = gross area of column, in.^{2} (mm^2)

F_{yc} = specified minimum yield strength of column, ksi (MPa)

P_{uc} = required column axial compressive strength, kips (a positive number) (N)

Z_b = plastic section modulus of the beam, in.^{3} (mm^3)

Z_c = plastic section modulus of the column, in.^{3} (mm^3)

z_b = minimum plastic section modulus at the Reduced Beam Section, in.^{3} (mm^3)

Exception: When columns conform to the requirements in Section 9.4, this requirement does not apply in the following two cases:

(a) Columns with $P_{uc} < 0.3F_{yc}A_g$ for all load combinations other than those determined using the Amplified Seismic Load that meet either of the following requirements:

(i) Columns used in a one-story building or the top story of a multistory building.

(ii) 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.

(b) Columns in any story that have 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

Column flanges at beam-to-column connections require lateral bracing only at the level of the top flanges of the beams when a column is shown to remain elastic outside of the Panel Zone. It shall be permitted to assume that the column remains elastic when the ratio calculated using Equation 9-3 is greater than 2.

When a column cannot be shown to remain elastic outside of the Panel Zone, the following requirements shall apply:

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

(2) Each column-flange lateral bracing shall be designed for a Required Strength that is equal to 2 percent of the nominal beam flange strength ($F_y b_f t_{bf}$).

(3) 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 bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral braces 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 the LRFD Specification, except that E shall be taken as the lesser of:

(a) The Amplified Seismic Load.

(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 9.7(1)(b) in addition to the second-order moment due to the resulting column flange displacement.

9.8. Lateral Bracing of Beams

Both flanges of beams shall be laterally braced directly or indirectly. The unbraced length between lateral braces shall not exceed $0.086 r_y E_s/F_y$. The Required Strength of lateral bracing shall be at least 2 percent of the beam flange Nominal Strength, $F_y b_f t_f$.

In addition, lateral braces 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. Where the design is based upon assemblies tested in accordance with Appendix S, the placement of lateral bracing for the beams shall be consistent with that used in the tests. The Required Strength of lateral bracing provided adjacent to plastic hinges shall be at least 6 percent of the expected Nominal Strength of the beam flange computed as $R_y F_y b_f t_f$. The required stiffness of all lateral bracing shall be determined in accordance with Equation C3-8 or C3-10, as applicable, of the LRFD Specification. In these equations, M_u shall be computed as $R_y Z F_y$.

9.9. Column Splices

Column splices shall comply with the requirements in Sections 8.4 and 7.3b. In addition, column splices in Special Moment Frames shall be located as described in Section 8.4a, and shall have a required flexural strength that is at least equal to R_y times the design flexural strength of the smaller column. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. Weld tabs shall be removed. Steel backing need not be removed unless required by the Engineer of Record. The required shear strength of column web splices shall be at least equal to $2M_{pc}/H$.

Exception: The Required Strength of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by inelastic analyses.

10. INTERMEDIATE MOMENT FRAMES (IMF)

10.1 Scope

Intermediate Moment Frames (IMF) 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. IMF shall meet the requirements in this Section.

10.2. Beam-to-Column Joints and Connections

10.2a. Requirements

All beam-to-column joints and connections used in the Seismic Load Resisting System shall satisfy the following three requirements:

(1) The connection must be capable of sustaining an Interstory Drift Angle of at least 0.02 radians.

(2) The flexural strength of the connection, determined at the column face, must equal at least 80 percent of the nominal plastic moment of the connected beam at an Interstory Drift Angle of 0.02 radians.

(3) The required shear strength V_u of the connection shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from the application of $2[1.1R_yF_yZ/$ distance between plastic hinge segments]. Alternatively, a lesser value of V_u is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of Load Combinations using the Amplified Seismic Load.

Connections that accommodate the required Interstory Drift Angle within the connection elements and provide the required flexural and shear strengths noted above are permitted, provided it can be demonstrated by analysis that the additional drift due to connection deformation can be accommodated by the building. Such analysis shall include effects of overall frame stability including second order effects.

10.2b. Conformance Demonstration

All beam-to-column joints and connections used in the Seismic Load Resisting System shall be demonstrated to satisfy the requirements of Section 10.2a. by one of the following:

(a) Use a connection prequalified for IMF in accordance with Appendix P.

(b) Provide qualifying cyclic test results in accordance with Appendix S. Results of at least two non-identical cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in research literature or documented tests performed for other projects that are demonstrated to represent project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

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

No additional requirements beyond the AISC LRFD Specification.

10.4. Beam and Column Limitations

No additional requirements beyond the AISC LRFD Specification.

10.5. Continuity Plates

Continuity Plates shall be provided to be consistent with the tested connection.

10.6. Column-Beam Moment Ratio

No additional requirements beyond the AISC LRFD Specification.

10.7. Beam-to-Column Connection Restraint

No additional requirements beyond the AISC LRFD Specification.

10.8. Lateral Bracing of Beams

No additional requirements beyond the AISC LRFD Specification.

10.9. Column Splices

Column splices shall comply with the requirements in Sections 8.4 and 7.3b.

11. ORDINARY MOMENT FRAMES (OMF)

11.1. Scope

Ordinary Moment Frames (OMF) are expected to withstand minimal 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

Beam-to-column connections shall be made with welds and/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 Load Resisting System shall be designed for a required flexural strength M_u that is at least equal to $1.1R_yM_p$ of the beam or girder or the maximum moment that can be delivered by the system, whichever is less.

(a) Where steel backing is used in connections with complete-joint-penetration (CJP) flange welds, steel backing and tabs shall be removed except that top-flange backing attached to the column by a continuous fillet weld on the edge below the CJP groove weld need not be removed. Removal of steel backing and tabs shall be as follows:

(i) Following the removal of backing, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The reinforcing fillet shall have a minimum leg size of 5/16-in. (8 mm).

(ii) Weld tab removal shall extend to within 1/8 in. (3 mm) of the base metal surface except at Continuity Plates where removal to within ¼ in. (6 mm) of the plate edge is acceptable. Edges of the weld tab shall be finished to a surface roughness value of 500 micro-in. (13 micrometers) or better. Grinding to a flush condition is not required. Gouges and notches are not permitted. The transitional slope of any area where gouges and notches have been removed shall not exceed 1:5. Material removed by grinding that extends more than 1/16 in. (2 mm) below the surface of the base metal shall be filled with weld metal. The contour of the weld at the ends shall provide a smooth transition, free of notches and sharp corners.

(b) Where weld access holes are provided, they shall be as shown in Figure 11-1. The weld access hole shall be ground smooth to a surface roughness value not to exceed 500 micro in. (13 micrometers), and shall be free of notches and gouges. Notches and gouges shall be repaired as required by the Engineer of Record.

(c) Double-sided partial-joint-penetration groove welds and double-sided fillet welds that resist tensile forces in connections shall be designed to resist a required force of $1.1R_yF_yA_g$ of the connected element or part. Single-sided partial-joint-penetration groove welds and single-sided fillet welds shall not be used to resist tensile forces in the connections.

(2) PR moment connections are permitted when the following requirements are met:

(a) Such connections shall provide for the Design Strength as specified in Section 11.2a.(1) above.

(b) The nominal flexural strength of the connection, M_n, shall be no less than 50 percent of M_p of the connected beam or column, whichever is less.

(c) The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.

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 the application of a moment of $2[1.1R_yF_yZ$ / distance between plastic hinge segments]. Alternatively, a lesser value of V_u is permitted if justified by analysis. 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 Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

No additional requirements beyond the AISC LRFD Specification.

11.4 Beam and Column Limitations

No additional requirements beyond the AISC LRFD Specification.

11.5 Continuity Plates

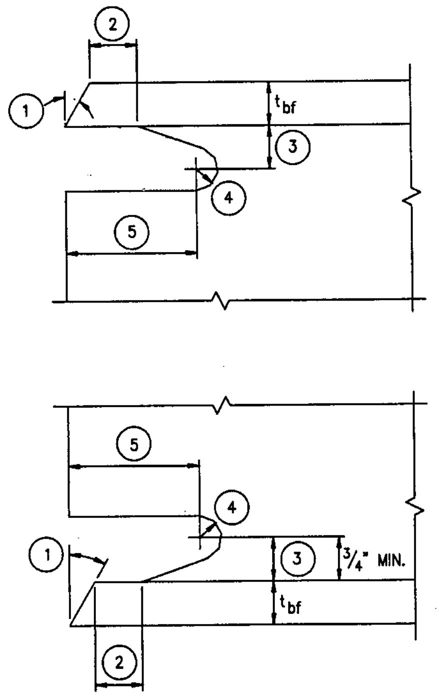
When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, Continuity Plates shall be provided to transmit beam flange forces to the column web or webs. Plates shall have a thickness greater than or equal to that of the beam flange 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. The Required Strength of these joints shall not be less than the Design Strength of the contact area of the plate with the column flange. The Required Strength of the welded joints of the Continuity Plates to the column web shall be the least 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.



Notes: 1. Bevel as required by AWS D1.1 for selected groove weld procedure.
2. Larger of t_{bf} or ½ in. (13 mm) (plus ½ t_{bf}, or minus ¼ t_{bf})
3. ¾ t_{bf} to t_{bf}, ¾ in. (19 mm) minimum (± ¼ in.) (± 6 mm)
4. 3/8 in. (10 mm) minimum radius (plus not limited, minus 0)
5. 3 t_{bf} (± ½ in.) (±13 mm)

Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°.

Fig. 11-1. Weld access hole detail (from FEMA 350, "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings").

11.6 Column-Beam Moment Ratio

No additional requirements beyond the AISC LRFD Specification.

11.7 Beam-to-Column Connection Restraint

No additional requirements beyond the AISC LRFD Specification.

11.8 Lateral Bracing of Beams

No additional requirements beyond the AISC LRFD Specification.

11.9 Column Splices

Column splices shall comply with the requirements in Section 8.4.

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 (20 m) and overall depth not to exceed 6 ft (1.8 m). 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 Load Resisting System shall have a special segment that is located between the quarter points of the span 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 the required vertical shear strength through the design flexural strength of the chord members and the design axial tensile and compressive strengths of the diagonal web members, when provided. 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 required 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

Members and connections of STMF, except those in the special segment defined in Section 12.2, shall have a Design Strength to resist the effects of load combinations stipulated by the Applicable Building Code, replacing the earthquake load term E with the lateral loads necessary to develop the expected vertical nominal shear strength in the special segment V_{ne} given as:

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

where

R_y = yield stress modification factor, see Section 6.2

M_{nc} = nominal flexural strength of the chord member of the special segment, kip-in. (N-mm)

$E_s I$ = flexural elastic stiffness of the chord members of the special segment, kip-in.^{2} (N-mm^2)

L = span length of the truss, in. (mm)

L_s = length of the special segment, in. (mm)

P_{nt} = nominal axial tension strength of diagonal members of the special segment, kips (N)

P_{nc} = nominal axial compression strength of diagonal members of the special segment, kips (N)

α = angle of diagonal members with the horizontal

12.5. Compactness

The width-thickness ratio of chord members shall not exceed the limiting λ_{ps} values from Table I-8-1. Diagonal web members within the special segment shall be made of flat bars.

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. The Required Strength of each lateral brace at the ends of and within the special segment shall be 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 Required Strength 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 $Kl/r \leq 5.87\sqrt{E_s / F_y}$.

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 stipulated by the Applicable Building Code including the Amplified Seismic Load. 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 compactness requirements in LRFD Specification Table B5.1 (i.e., $\lambda<\lambda_p$) and the following requirements:

(1) The width-thickness ratio of angle legs shall comply with λ_{ps} in Table I-8-1.

(2) I-shaped members and channels shall comply with λ_{ps} in Table I-8-1.

(3) Round HSS shall have an outside diameter to wall thickness ratio conforming to Table I-8-1 unless the round HSS wall is stiffened.

(4) Rectangular HSS shall have a flat width to wall thickness ratio conforming to Table I-8-1 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

13.3a. 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 $R_yF_yA_g$.

(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 Section J4, 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 the brace will buckle, the required flexural strength of the connection shall be equal to $1.1R_yM_p$ 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 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 load combinations stipulated by the Applicable Building Code, assuming that the bracing is not present.

(3) A beam that is intersected by braces shall be designed to resist the effects of load combinations stipulated by the Applicable Building Code, 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 R_yP_y for the brace in tension and a maximum of 0.3 times ϕ_cP_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 $F_yb_ft_{bf}$.

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:

Width-thickness ratios of stiffened and unstiffened compression elements of columns shall meet the requirements for bracing members in Section 13.2d.

In addition to meeting the requirements in Section 8.4, 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 Concentrically 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. Strength

The Required Strength of the members and connections, other than brace connections, in OCBFs shall be determined using the load combinations stipulated by the Applicable Building Code, including the Amplified Seismic Load. The Required Strength of brace connections is the expected tensile strength of the brace, determined as $R_y F_y A_g$. Braces with Kl/r greater than $4.23\sqrt{E_s / F_y}$ shall not be used in V or inverted-V configurations.

15. ECCENTRICALLY BRACED FRAMES (EBF)

15.1. Scope

Eccentrically Braced Frames (EBFs) 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

Links shall comply with the width-thickness ratios in Table I-8-1.

The specified minimum yield stress of steel used for Links shall not exceed 50 ksi (345 MPa)

The web of a Link shall be single thickness without doubler-plate reinforcement and without web penetrations.

Except as limited below, the required shear strength of the Link V_u shall not exceed the design shear strength of the Link ϕV_n,

where:

ϕ = 0.9
V_n = Nominal shear strength of the Link, equal to the lesser of V_p or $2M_p/e$, kips (N)
V_p = $0.6F_yA_w$, kips (N)
e = Link length, in. (mm)
A_w = $(d_b\text{-}2t_f)t_w$

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.

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:

$$\phi = 0.9$$

$$V_{pa} = V_p \sqrt{1 - \left(P_u / P_y\right)^2} \qquad (15\text{-}1)$$

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

(2) The length of the Link shall not exceed:

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

nor

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

where:
$A_w = (d_b - 2t_f)t_w$
$\rho' = P_u/V_u$

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:

(a) 0.08 radians for Links of length $1.6M_p/V_p$ or less.
(b) 0.02 radians for Links of length $2.6M_p/V_p$ or greater.
(c) 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

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. (10 mm), whichever is larger, where b_f and t_w are the Link flange width and Link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

(a) 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.

(b) 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.

(c) 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.

(d) Intermediate web stiffeners are not required in Links of lengths greater than $5M_p/V_p$.

(e) Intermediate Link web stiffeners shall be full depth. For Links that are less than 25 in. (635 mm) 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 3/8 in. (10 mm), whichever is larger, and the width shall be not less than $(b_f/2)$-t_w. For Links that are 25 in. (635 mm) in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The Required Strength of fillet welds connecting a Link stiffener to the Link web is $A_{st}F_y$, where A_{st} is the area of the stiffener. The Required Strength of fillet welds fastening the stiffener to the flanges is $A_{st}F_y/4$.

15.4. Link-to-Column Connections

Link-to-column connections must be capable of sustaining the maximum Link Rotation Angle based on the length of the Link, as specified in Section 15.2. The strength of the connection, measured at the column face, must equal at least the nominal shear strength of the Link, V_n, as specified in Section 15.2 at the maximum Link Rotation Angle.

Link-to-column connections shall be demonstrated to satisfy the above requirements by one of the following:

(a) Use a connection Prequalified for EBF in accordance with Appendix P.

(b) Provide qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in research literature or documented tests performed for other projects that are demonstrated to represent project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

Exception: 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 15.6. Full depth stiffeners as required in Section 15.3 shall be placed at the Link-to-reinforcement interface.

15.5. Lateral Bracing of Link

Lateral bracing shall be provided at both the top and bottom Link flanges at the ends of the

Link. The Required Strength of end lateral bracing of Links is 6 percent of the expected Nominal Strength of the Link flange computed as $R_yF_yb_ft_f$.

15.6. Diagonal Brace and Beam Outside of Link

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_yV_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.

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_yV_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 bracing where analysis indicates that support is necessary to maintain the stability of the beam. Lateral bracing 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 $F_yb_ft_f$.

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.

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 15.6. 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.

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 $F_yb_ft_f$ 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 load combinations as stipulated by the Applicable Building Code, 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 15.2.

16. QUALITY ASSURANCE

The general requirements and responsibilities for performance of a quality assurance plan shall be in accordance with the requirements of the Authority Having Jurisdiction 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 control program and qualifications, such as participation in a recognized quality certification program, shall be considered when establishing a quality control 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:

(1) 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 also be performed, but shall not serve to replace visual inspection.

(2) All complete-joint-penetration and partial-joint-penetration groove welded joints that are subjected to net tensile forces as part of the Seismic Load 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 Authority Having Jurisdiction.

When welds from web doubler plates or Continuity Plates occur in the *k-Area* of rolled steel columns, the *k-Area* adjacent to the welds shall be inspected after fabrication, as required by the Engineer of Record, using approved nondestructive methods conforming to AWS D1.1.

APPENDIX P

PREQUALIFICATION OF BEAM-COLUMN AND LINK-TO-COLUMN CONNECTIONS

P1. SCOPE

This appendix contains minimum requirements for prequalification of beam-to-column moment connections in Special Moment Frames (SMFs) and Intermediate Moment Frames (IMFs), and link-to-column connections in Eccentrically Braced Frames (EBFs). Prequalified Connections are permitted to be used, within the applicable limits of prequalification, without the need for further qualifying cyclic tests.

P2. GENERAL REQUIREMENTS

P2.1. Basis for Prequalification

Connections shall be Prequalified based on test data satisfying Section P3, supported by analytical studies and design models. The combined body of evidence for prequalification must be sufficient to assure that the connection can supply the required Interstory Drift Angle for SMF and IMF systems, or the required Link Rotation Angle for EBFs, on a consistent and reliable basis within the specified limits of prequalification. All applicable limit states for the connection that affect the stiffness, strength and deformation capacity of the connection and the Seismic Load Resisting System must be identified. These include fracture related limit states, stability related limit states, and all other limit states pertinent for the connection under consideration. The effect of design variables listed in Section P4 shall be addressed for connection prequalification.

P2.2. Authority for Prequalification

Prequalification of a connection and the associated limits of prequalification shall be established by a Connection Prequalification Review Panel (CPRP) approved by the Authority Having Jurisdiction.

P3. TESTING REQUIREMENTS

Data used to support connection prequalification shall be based on tests conducted in accordance with Appendix S. The CPRP shall determine the number of tests and the variables considered by the tests for connection prequalification. The CPRP shall also provide the same information when limits are to be changed for a previously prequalified connection. A sufficient number of tests shall be performed on enough non-identical specimens to demonstrate that the connection has the ability and reliability to undergo the required Interstory Drift Angle for SMFs and IMFs and the required Link Rotation Angle for EBFs, where the Link is adjacent to columns. For connections that are already Prequalified Connections, and the limits of prequalification are being changed, additional non-identical specimens shall be tested prior to changing prequalification limits. The limits on member sizes for prequalification shall not exceed the limits specified in Appendix S, Section S5.2.

P4. PREQUALIFICATION VARIABLES

In order to be Prequalified, the effect of the following variables on connection performance shall be considered. Limits on the permissible values for each variable shall be established by the CPRP for the Prequalified Connection.

(1) Beam or Link parameters:
- (a) Cross-section shape: wide flange, box, or other.
- (b) Cross-section fabrication method: rolled shape, welded shape, or other.
- (c) Depth.
- (d) Weight per foot.
- (e) Flange thickness.
- (f) Material specification.
- (g) Span-to-depth ratio (for SMF or IMF), or Link length (for EBF).
- (h) Width thickness ratio of cross-section elements.
- (i) Lateral bracing.
- (j) Other parameters pertinent to the specific connection under consideration.

(2) Column parameters:
- (a) Cross-section shape: wide flange, box, or other.
- (b) Cross-section fabrication method: rolled shape, welded shape, or other.
- (c) Column orientation with respect to beam or Link: beam or Link is connected to column flange, beam or Link is connected to column web, beams or Links are connected to both the column flange and web, or other.
- (d) Depth.
- (e) Weight per foot.
- (f) Flange thickness.
- (g) Material specification.
- (h) Width-thickness ratio of cross-section elements.
- (i) Lateral bracing.
- (j) Other parameters pertinent to the specific connection under consideration.

(3) Beam (or Link) – Column Relations:
- (a) Panel zone strength.
- (b) Doubler plate attachment details.
- (c) Column-beam (or Link) moment ratio.

(4) Continuity Plates:
- (a) Identification of conditions under which Continuity Plates are required.
- (b) Thickness, width and depth.
- (c) Attachment details.

(5) Welds:
- (a) Weld type: CJP, PJP, fillet, or plug.

(b) Filler metal strength and toughness.
(c) Details and treatment of weld backing and weld tabs.
(d) Weld access holes: size, geometry and finish.
(e) Welding quality control and quality assurance.
(f) Other parameters pertinent to the specific connection under consideration.

(6) Bolts:
(a) Bolt diameter.
(b) Bolt Grade: ASTM A325, A490, or other.
(c) Installation requirements: pretensioned, snug tight, or other.
(d) Hole type: standard, oversize, short-slot, long-slot, or other.
(e) Hole fabrication method: drilling, punching, sub-punching and reaming, or other.
(f) Other parameters pertinent to the specific connection under consideration.

(7) Additional Connection Details: All variables pertinent to the specific connection under consideration, as established by the CPRP.

P5. DESIGN PROCEDURE

A comprehensive design procedure must be available for a Prequalified Connection. The design procedure must address all applicable limit states within the limits of prequalification.

P6. PREQUALIFICATION RECORD

A Prequalified Connection shall be provided with a written prequalification record with the following information:

(1) General description of the Prequalified Connection and drawings that clearly identify key features and components of the connection.
(2) Description of the expected behavior of the connection in the elastic and inelastic ranges of behavior, intended location(s) of inelastic action, and a description of limit states controlling the strength and deformation capacity of the connection.
(3) Listing of systems for which connection is Prequalified: SMF, IMF or EBF.
(4) Listing of limits for all prequalification variables listed in Section P4.
(5) A detailed description of the design procedure for the connection, as required in Section P5.
(6) A list of references of test reports, research reports and other publications that provided the basis for prequalification.
(7) Summary of material strengths
(8) Summary of quality control procedures.

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 beam-to-column connection or a Link-to-column connection satisfies the requirements for strength and Interstory Drift Angle or Link Rotation Angle in these Provisions. Alternative testing requirements are permitted when approved by the Engineer of Record and the Authority Having Jurisdiction.

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.

θ Interstory Drift Angle (S6)
γ Link Rotation Angle (S6)

S3. DEFINITIONS

Complete Loading Cycle. A cycle of rotation taken from zero force to zero force, including one positive and one negative peak.

Interstory Drift Angle. Interstory displacement divided by story height, radians.

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 on an analysis of Test Specimen deformations. Sources of inelastic rotation include yielding of members, yielding of connection elements and connectors, and slip between members and connection elements. For beam-to-column moment connections in Moment Frames, inelastic rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the beam with the centerline of the column. For Link-to-column connections in Eccentrically Braced Frames, inelastic rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the Link with the face of the column.

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

S4. TEST SUBASSEMBLAGE REQUIREMENTS

The Test Subassemblage shall replicate as closely as is practical the conditions that will occur in the Prototype during earthquake loading. The Test Subassemblage 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 Subassemblage is permitted near load application or reaction points as needed to provide lateral stability of the Test Subassemblage. Additional lateral bracing of the Test Subassemblage 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

The size of the beam or Link used in the Test Specimen shall be within the following limits:

(1) The depth of the test beam or Link shall be no less than 90 percent of the depth of the Prototype beam or Link.

(2) 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.

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. In addition, the depth of the test column shall be no less than 90 percent of the depth of the Prototype column.

Extrapolation beyond the limitations stated in this Section shall be permitted subject to qualified peer review and approval by the Authority Having Jurisdiction.

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 of the beam shall not be more than 15 percent below R_yF_y for the grade of steel to be used for the corresponding elements of the Prototype. Columns and connection elements with a tested yield stress shall not be more than 15 percent above or below R_yF_y for the grade of steel to be used for the corresponding elements of the Prototype. R_yF_y shall be determined in accordance with Section 6.2.

S5.6. Welds

Welds on the Test Specimen shall satisfy the following requirements:

(1) Welding shall be performed in strict conformance with 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, ASTM F1852) 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 (pretensioned or other) and faying surface preparation (no specified slip resistance, Class A, B, or C 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 Section S6.2 for beam-to-column moment connections in Moment Frames, and according to the requirements prescribed in Section S6.3 for link-to-column connections in Eccentrically Braced Frames.

Loading sequences other than those specified in Sections S6.2 and S6.3 may be used when they are demonstrated to be of equivalent or greater severity.

S6.2. Loading Sequence for Beam-to-Column Moment Connections

Qualifying cyclic tests of beam-to-column moment connections in Moment Frames shall be conducted by controlling the Interstory Drift Angle, θ, imposed on the Test Specimen, as follows:

(1) 6 cycles at $\theta = 0.00375$ rad.
(2) 6 cycles at $\theta = 0.005$ rad.
(3) 6 cycles at $\theta = 0.0075$ rad.
(4) 4 cycles at $\theta = 0.01$ rad.
(5) 2 cycles at $\theta = 0.015$ rad.
(6) 2 cycles at $\theta = 0.02$ rad.
(7) 2 cycles at $\theta = 0.03$ rad.
(8) 2 cycles at $\theta = 0.04$ rad.

Continue loading at increments of $\theta = 0.01$ radians, with two cycles of loading at each step.

S6.3. Loading Sequence for Link-to-Column Connections

Qualifying cyclic tests of link-to-column moment connections in Eccentrically Braced Frames shall be conducted by controlling the Link Rotation Angle, γ, imposed on the Test Specimen, as follows:

(1) 3 cycles at $\gamma = 0.0025$ rad.
(2) 3 cycles at $\gamma = 0.005$ rad.
(3) 3 cycles at $\gamma = 0.01$ rad.
(4) 2 cycles at $\gamma = 0.02$ rad.
(5) 2 cycles at $\gamma = 0.03$ rad.
(6) 2 cycles at $\gamma = 0.04$ rad.

Continue loading at increments of $\gamma = 0.01$ radians, with two cycles of loading at each step.

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/A6M, 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 Authority Having Jurisdiction 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 Subassemblage, 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 Interstory Drift Angle for beam-to-column moment connections; or a plot of Link shear force versus Link Rotation Angle for link-to-column connections. For beam-to-column connections, the beam moment and the Interstory Drift Angle shall be computed with respect to the centerline of the column.

(7) The Interstory Drift Angle and 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 chronological 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

The Test Specimen must satisfy the strength and Interstory Drift Angle or Link Rotation Angle requirements of these Provisions for the SMF, IMF, or EBF connection, as applicable. The Test Specimen must sustain the required Interstory Drift Angle or Link Rotation Angle for at least one complete loading cycle.

APPENDIX X
WELD METAL / WELDING PROCEDURE SPECIFICATION TOUGHNESS VERIFICATION TEST

***Preamble**: This appendix provides a procedure for qualifying the weld metal toughness and is included on an interim basis pending adoption of such a procedure by AWS or other accredited organization.*

X1. Scope

This appendix provides a standard method for qualification testing of weld filler metals required to have specified notch toughness for service in specified joints in steel Moment Frames for seismic applications.

Testing of weld metal to be used in production shall be performed by filler metal manufacturer's production lot, as defined in AWS A5.01, *Filler Metal Procurement Guidelines,* as follows:

(1) Class C3 for SMAW electrodes,

(2) Class S2 for GMAW-S and SAW electrodes,

(3) Class T4 for FCAW and GMAW-C, or

(4) Class F2 for SAW fluxes.

Alternatively, filler metal manufacturers approved for production of products meeting the above requirements, under a program acceptable to the Engineer, need not conduct the mechanical A5 tests or the Weld Metal / Weld Procedure Specification (WPS) Toughness Verification Test, or require lot control for each lot, and may rely upon the Manufacturer's certifications that the product meets the specified performance requirements.

X2. Test Conditions

Tests shall be conducted at the range of heat inputs for which the weld filler metal will be qualified under the WPS. It is recommended that tests be conducted at the Low Heat Input Level and High Heat Input Level indicated in Table I-X-1.

Table I-X-l WPS Toughness Verification Test
Welding and Preheat Conditions

Cooling Rate	Heat Input	Preheat °F (°C)	Interpass °F (°C)
Low Heat Input Test	30 kJ/in. (1.2 kJ/mm)	70 ± 25 (21 ± 14)	200 ± 50 (93 ± 28)
High Heat Input Test	80 kJ/in. (3.1 kJ/mm)	300 ± 25 (149 ± 14)	500 ± 50 (260 ± 28)

Alternatively, the filler metal manufacturer or Contractor may elect to test a wider or narrower range of heat inputs and interpass temperatures. The range of heat inputs and interpass temperatures tested shall be clearly stated on the test reports and user data sheets. Regardless of the method of selecting test heat input, the WPS, as used by the contractor, shall fall within the range of heat inputs and interpass temperatures tested.

X3. Test Specimens

Two test plates, one for each heat input level shall be used, and five Charpy V-Notch (CVN) test specimens shall be made per plate. Each plate shall be steel, of any AISC-listed structural grade. The test plate shall be 3/4 in. (19 mm) thick with a 1/2-inch (13 mm) root opening and 45° included groove angle. The test plate and specimens shall be as shown in Figure 2A in AWS A5.20-95, or as in Figure 5 in AWS A5.29-98. Except for the root pass, a minimum of two passes per layer shall be used to fill the width.

All test specimens shall be taken from near the centerline of the weld at the mid-thickness location, in order to minimize dilution effects. CVN specimens shall be prepared in accordance with AWS B4.0-92, *Standard Methods for Mechanical Testing of Welds,* Section A3. The test assembly shall be restrained during welding, or preset at approximately 5 degrees to prevent warpage in excess of 5 degrees. A welded test assembly that has warped more than 5 degrees shall be discarded. Welded test assemblies shall not be straightened.

The test assembly shall be tack welded and heated to the specified preheat temperature, measured by temperature indicating crayons or surface temperature thermometers one inch from the center of the groove at the location shown in the figures cited above. Welding shall continue until the assembly has reached the interpass temperature prescribed in Table I-X-1. The interpass temperature shall be maintained for the remainder of the weld. Should it be necessary to interrupt welding, the assembly shall be allowed to cool in air. The assembly shall then be heated to the prescribed interpass temperature before welding is resumed.

X4. Acceptance Criteria

The lowest and highest CVN toughness values obtained from the five specimens from a single test plate shall be disregarded. Two of the remaining three values shall equal, or exceed, the specified toughness of 40 ft-lbf (54 J) energy level at the testing temperature. One of the three may be lower, but not lower than 30 ft-lbf (41 J), and the average of the three shall not be less than the required 40 ft-lbf (54 J) energy level. All test samples shall meet the notch toughness requirements for the electrodes as provided in Section 7.3b.

PART II. COMPOSITE STRUCTURAL STEEL AND REINFORCED CONCRETE BUILDINGS

Glossary

The following glossary terms are applicable to Part II and are in addition to those given in the Part I Glossary.

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

Collector Element. 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.

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.

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.

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.

1. SCOPE

These Provisions are intended for the design and construction of composite structural steel and reinforced concrete members and connections in the Seismic Load 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.

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 Load Resisting System shall have a Design Strength as required in the LRFD Specification 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 SPECIFICATIONS, 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 Society of Civil Engineers
Standard for the Structural Design of Composite Slabs, ASCE 3-91

3. SEISMIC DESIGN CATEGORIES

The Required Strength and other seismic provisions for Seismic Design Categories, Seismic Use Groups or Seismic Zones and the limitations on height and irregularity shall be as stipulated in the Applicable Building Code .

4. LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS

The loads and load combinations shall be as stipulated by the Applicable Building Code (see Glossary). Where Amplified Seismic Loads are required by these provisions, the horizontal earthquake load E (as defined in the Applicable Building Code) shall be multiplied by the overstrength factor Ω_o prescribed by the Applicable Building Code. In the absence of a specific

TABLE II-4-1

System Overstrength Factor, Ω_o

Seismic Load Resisting System	Ω_o
All moment-frame systems meeting Part II requirements	3
All Eccentrically Braced Frames (EBF) and wall systems meeting Part II requirements	2½
All other systems meeting Part II requirements	2

definition of Ω_o, the value for Ω_o shall be as listed in Table II-4-1.

5. MATERIALS

5.1. Structural Steel

Structural steel used in composite Seismic Load Resisting Systems shall meet the requirements in LRFD Specification Section A3.1a. Structural steel used in the composite Seismic Force Resisting Systems described in Sections 8, 9, 13, 14, 16 and 17 shall also meet the requirements in Part I Section 6.

5.2. Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite Seismic Load Resisting Systems shall meet the requirements in ACI 318, excluding Chapters 21 and 22, and the following requirements:

(1) The specified minimum compressive strength of concrete in composite members shall equal or exceed 2.5 ksi (17 MPa).
(2) For the purposes of determining the Nominal Strength of composite members, f'_c shall not be taken as greater than 10 ksi (69 MPa) for normal-weight concrete nor 4 ksi (28 MPa) for lightweight concrete.

Concrete and steel reinforcement used in the composite Seismic Load Resisting Systems described in Sections 8, 9, 13, 14, 16, and 17 shall also meet the requirements in ACI 318 Chapter 21.

6. COMPOSITE MEMBERS

6.1 Scope

The design of composite members in the Seismic Load Resisting Systems described in Sections 8 through 17 shall meet the requirements in this Section and the material requirements in Section 5.

6.2. Composite Floor and Roof Slabs

The design of composite floor and roof slabs shall meet the requirements of ASCE 3. Composite slab diaphragms shall meet the requirements in this Section.

Details shall be designed to transfer forces between the diaphragm and Boundary Members, Collector Elements, and elements of the horizontal framing system.

The nominal shear strength of composite diaphragms and concrete-filled steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with ACI 318 excluding Chapter 22. Alternatively, the composite diaphragm design shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

6.3. Composite Beams

Composite Beams shall meet the requirements in LRFD Specification Chapter I. Composite Beams that are part of C-SMF as described in Section 9 shall also meet the following requirements:

(1) The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed:

$$\frac{Y_{con} + d_b}{1 + \left(\dfrac{1,700\,F_y}{E_s}\right)} \tag{6-1}$$

where

Y_{con} = distance from the top of the steel beam to the top of concrete, in. (mm)
d_b = depth of the steel beam, in. (mm)
F_y = specified minimum yield strength of the steel beam, ksi (MPa)
E_s = modulus of elasticity of the steel beam, ksi (MPa)

(2) Beam flanges shall meet the requirements in Part I Section 9.4, except when fully reinforced-concrete-encased compression elements have a reinforced concrete cover of at least 2 in. (50 mm) and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement shall meet the requirements in ACI 318 Section 21.3.3.

6.4. Reinforced-Concrete-Encased Composite Columns

This Section is applicable to columns that: (1) consist of reinforced-concrete-encased structural steel sections with a structural steel area that comprises at least 4 percent of the total composite-column cross-section; and (2) meet the additional limitations in LRFD Specification Section I2.1. Such columns shall meet the requirements in LRFD

Specification Chapter I, except as modified in this Section. Additional requirements, as specified for intermediate and special seismic systems in Sections 6.4b and 6.4c, shall apply as required in the descriptions of the composite seismic systems in Sections 8 through 17.

Columns that consist of reinforced-concrete-encased structural steel sections with a structural steel area that comprises less than 4 percent of the total composite-column cross-section shall meet the requirements for reinforced concrete columns in ACI 318 except as modified for:

(1) The steel shape shear connectors in Section 6.4a.2.

(2) The contribution of the reinforced-concrete-encased structural steel section to the strength of the column as provided in ACI 318.

(3) The seismic requirements for reinforced concrete columns as specified in the description of the composite seismic systems in Sections 8 through 17.

6.4a. Ordinary Seismic System Requirements

The following requirements for Reinforced-Concrete-Encased Composite Columns are applicable to all composite systems:

(1) The nominal shear strength of the column shall be determined as the nominal shear strength of the structural shape plus the nominal shear strength that is provided by the tie reinforcement in the reinforced-concrete encasement. The nominal shear strength of the structural steel section shall be determined in accordance with LRFD Specification Section F2. The nominal shear strength of the tie reinforcement shall be determined in accordance with ACI 318 Sections 11.5.6.2 through 11.5.6.9. In ACI 318 Sections 11.5.6.5 and 11.5.6.9, the dimension b_w shall equal the width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear. The nominal shear strength shall be multiplied by ϕ_v equal to 0.75 to determine the design shear strength.

(2) Composite Columns that are designed to share the applied loads between the structural steel section and reinforced concrete shall have shear connectors that meet the following requirements:

(a) If an external member is framed directly to the structural steel section to transfer a vertical reaction V_u, shear connectors shall be provided to transfer the force $V_u(1 - A_sF_y/P_n)$ between the structural steel section and the reinforced concrete, where A_s is the area of the structural steel section, F_y is the specified minimum yield strength of the structural steel section, and P_n is the nominal compressive strength of the Composite Column.

(b) If an external member is framed directly to the reinforced concrete to transfer a vertical reaction V_u, shear connectors shall be provided to transfer the force

$V_u A_s F_y / P_n$ between the structural steel section and the reinforced concrete, where A_s, F_y and P_n are as defined above.

(c) The maximum spacing of shear connectors shall be 16 in. (406 mm) with attachment along the outside flange faces of the embedded shape.

(3) The maximum spacing of transverse ties shall be the least of the following:

(a) one-half the least dimension of the section
(b) 16 longitudinal bar diameters
(c) 48 tie diameters

Transverse ties shall be located vertically within one-half the tie spacing above the top of the footing or lowest beam or slab in any story and shall be spaced as provided herein within one-half the tie spacing below the lowest beam or slab framing into the column.

Transverse bars shall have a diameter that is not less than one-fiftieth of greatest side dimension of the composite member, except that ties shall not be smaller than No. 3 bars and need not be larger than No. 5 bars. Alternatively, welded wire fabric of equivalent area is permitted as transverse reinforcement except when prohibited for intermediate and special systems.

(4) All Load-Carrying Reinforcement shall meet the detailing and splice requirements in ACI 318 Sections 7.8.1 and 12.17. Load-Carrying Reinforcement shall be provided at every corner of a rectangular cross-section. The maximum spacing of other load carrying or restraining longitudinal reinforcement shall be one-half of the least side dimension of the composite member.

(5) Splices and end bearing details for reinforced-concrete-encased structural steel sections shall meet the requirements in the LRFD Specification and ACI 318 Section 7.8.2. If adverse behavioral effects due to the abrupt change in member stiffness and nominal tensile strength occur when reinforced-concrete encasement of a structural steel section is terminated, either at a transition to a pure reinforced concrete column or at the Column Base, they shall be considered in the design.

6.4b. Intermediate System Requirements

Reinforced-Concrete-Encased Composite Columns in intermediate seismic systems shall meet the following requirements in addition to those in Section 6.4a:

(1) The maximum spacing of transverse bars at the top and bottom shall be the least of the following:

(a) one-half the least dimension of the section
(b) 8 longitudinal bar diameters
(c) 24 tie bar diameters

(d) 12 in. (305 mm)

These spacings shall be maintained over a vertical distance equal to the greatest of the following lengths, measured from each joint face and on both sides of any section where flexural yielding is expected to occur:

(a) one-sixth the vertical clear height of the column
(b) the maximum cross-sectional dimension
(c) 18 in. (457 mm)

(2) Tie spacing over the remaining column length shall not exceed twice the spacing defined above.

(3) Welded wire fabric is not permitted as transverse reinforcement in intermediate seismic systems.

6.4c. Special Seismic System Requirements

Reinforced-concrete-encased columns for special seismic systems shall meet the following requirements in addition to those in Sections 6.4.a. and 6.4.b.:

(1) The required axial strength for Reinforced-Concrete-Encased Composite Columns and splice details shall meet the requirements in Part I Section 8.

(2) Longitudinal Load-Carrying Reinforcement shall meet the requirements in ACI 318 Section 21.4.3.

(3) Transverse reinforcement shall be hoop reinforcement as defined in ACI 318 Chapter 21 and shall meet the following requirements:

(a) The minimum area of tie reinforcement A_{sh} shall meet the following requirement:

$$A_{sh} = 0.09\, h_{cc} s \left(1 - \frac{F_y A_s}{P_n}\right)\left(\frac{f_c'}{F_{yh}}\right) \tag{6-2}$$

where

h_{cc} = cross-sectional dimension of the confined core measured center-to-center of the tie reinforcement, in. (mm)
s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, in. (mm)
F_y = specified minimum yield strength of the structural steel core, ksi (MPa)
A_s = cross-sectional area of the structural core, in.^{2} (mm^2)
P_n = nominal axial compressive strength of the Composite Column calculated in accordance with the LRFD Specification, kips (N)
f'_c = specified compressive strength of concrete, ksi (MPa)

F_{yh} = specified minimum yield strength of the ties, ksi (MPa)

Equation 6-2 need not be satisfied if the Nominal Strength of the reinforced-concrete-encased structural steel section alone is greater than 1.0*D*+0.5*L*.

(b) The maximum spacing of transverse reinforcement along the length of the column shall be the lesser of 6 longitudinal load-carrying bar diameters and 6 in. (152 mm)

(c) When specified in Sections 6.4c.(4), 6.4c.(5) or 6.4c.(6), the maximum spacing of transverse reinforcement shall be the lesser of one-fourth the least member dimension and 4 in. (102 mm). For this reinforcement, cross ties, legs of overlapping hoops, and other confining reinforcement shall be spaced not more than 14 in. on center in the transverse direction.

(4) Reinforced-Concrete-Encased Composite Columns in Braced Frames with axial compression forces that are larger than 0.2 times P_o shall have transverse reinforcement as specified in Section 6.4c.(3)(c) over the total element length. This requirement need not be satisfied if the Nominal Strength of the reinforced-concrete-encased steel section alone is greater than 1.0*D*+0.5*L*.

(5) Composite Columns supporting reactions from discontinued stiff members, such as walls or Braced Frames, shall have transverse reinforcement as specified in Section 6.4c.(3)(c) over the full length beneath the level at which the discontinuity occurs if the axial compression force exceeds 0.1 times P_o. Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the reinforced-concrete-encased structural steel section and longitudinal reinforcement. This requirement need not be satisfied if the Nominal Strength of the reinforced-concrete-encased structural steel section alone is greater than 1.0*D*+0.5*L*.

(6) Reinforced-Concrete-Encased Composite Columns that are used in C-SMF shall meet the following requirements:

(a) Transverse reinforcement shall meet the requirements in Section 6.4c(3)(c) at the top and bottom of the column over the region specified in Section 6.4b.

(b) The strong-column/weak-beam design requirements in Section 9.5 shall be satisfied. Column Bases shall be detailed to sustain inelastic flexural hinging.

(c) The minimum required shear strength of the column shall meet the requirements in ACI 318 Section 21.4.5.1.

(7) When the column terminates on a footing or mat foundation, the transverse reinforcement as specified in this section shall extend into the footing or mat at least 12 in. (305 mm). When the column terminates on a wall, the transverse reinforcement shall extend into the wall for at least the length required to develop full yielding in the reinforced-concrete-encased structural steel section and longitudinal reinforcement.

(8) Welded wire fabric is not permitted as transverse reinforcement for special seismic systems.

6.5. Concrete-Filled Composite Columns

This Section is applicable to columns that: (1) consist of concrete-filled steel rectangular or circular hollow structural sections (HSS) with a structural steel area that comprises at least 4 percent of the total composite-column cross-section; and (2) meet the additional limitations in LRFD Specification Section I2.1. Such columns shall be designed to meet the requirements in LRFD Specification Chapter I, except as modified in this Section.

The design shear strength of the Composite Column shall be the design shear strength of the structural steel section alone.

In the special seismic systems described in Sections 9, 13 and 14, members and column splices for Concrete-Filled Composite Columns shall also meet the requirements in Part I Section 8.

Concrete-Filled Composite Columns used in C-SMF shall meet the following additional requirements:

(1) The minimum required shear strength of the column shall meet the requirements in ACI 318 Section 21.4.5.1.

(2) The strong-column/weak-beam design requirements in Section 9.5 shall be met. Column Bases shall be designed to sustain inelastic flexural hinging.

(3) The minimum wall thickness of concrete-filled rectangular HSS shall equal

$$b\sqrt{F_y/(2E_s)} \tag{6-3}$$

for the flat width b of each face, where b is as defined in LRFD Specification Table B5.1.

7. COMPOSITE CONNECTIONS

7.1. Scope

This Section is applicable to connections in buildings that utilize composite or dual steel and concrete systems wherein seismic force is transferred between structural steel and reinforced concrete components.

Composite connections shall be demonstrated to have Design Strength, ductility and toughness that is comparable to that exhibited by similar structural steel or reinforced concrete connections that meet the requirements in Part I and ACI 318, respectively. Methods for calculating the connection strength shall meet the requirements in this Section.

7.2. General Requirements

Connections shall have adequate deformation capacity to resist the critical Required Strengths at the Design Story Drift. Additionally, connections that are required for the lateral stability of the building under seismic forces shall meet the requirements in Sections 8 through 17 based upon the specific system in which the connection is used. When the Required Strength is based upon nominal material strengths and nominal member dimensions, the determination of the required connection strength shall account for any effects that result from the increase in the actual Nominal Strength of the connected member.

7.3. Nominal Strength of Connections

The Nominal Strength of connections in composite Structural Systems shall be determined on the basis of rational models that satisfy both equilibrium of internal forces and the strength limitation of component materials and elements based upon potential limit states. Unless the connection strength is determined by analysis and testing, the models used for analysis of connections shall meet the following requirements:

(1) When required, force shall be transferred between structural steel and reinforced concrete through direct bearing of headed shear studs or suitable alternative devices, by other mechanical means, by shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer, or by a combination of these means. Any potential bond strength between structural steel and reinforced concrete shall be ignored for the purpose of the connection force transfer mechanism.

(2) The nominal bearing and shear-friction strengths shall meet the requirements in ACI 318 Chapters 10 and 11, except that the strength reduction (resistance) factors shall be as given in ACI 318. Unless a higher strength is substantiated by cyclic testing, the nominal bearing and shear-friction strengths shall be reduced by 25 percent for the composite seismic systems described in Sections 9, 13, 14, 16, and 17.

(3) The Design Strengths of structural steel components in composite connections, as determined in Part I and the LRFD Specification, shall equal or exceed the Required Strengths. Structural steel elements that are encased in confined reinforced concrete are permitted to be considered to be braced against out-of-plane buckling. Face Bearing Plates consisting of stiffeners between the flanges of steel beams are required when beams are embedded in reinforced concrete columns or walls.

(4) The nominal shear strength of reinforced-concrete-encased steel Panel Zones in beam-to-column connections shall be calculated as the sum of the Nominal Strengths of the structural steel and confined reinforced concrete shear elements as determined in Part I Section 9.3 and ACI 318 Section 21.5, respectively. The strength reduction (resistance) factors for reinforced concrete shall be as given in ACI 318.

(5) Reinforcement shall be provided to resist all tensile forces in reinforced concrete

components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as appropriate, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with ACI 318 Chapter 12. Additionally, development lengths for the systems described in Sections 9, 13, 14, 16 and 17 shall meet the requirements in ACI 318 Section 21.5.4. Connections shall meet the following additional requirements:

(a) When the slab transfers horizontal diaphragm forces, the slab reinforcement shall be designed and anchored to carry the in-plane tensile forces at all critical sections in the slab, including connections to collector beams, columns, braces and walls.

(b) For connections between structural steel or Composite Beams and reinforced concrete or Reinforced-Concrete-Encased Composite Columns, transverse hoop reinforcement shall be provided in the connection region to meet the requirements in ACI 318 Section 21.5, except for the following modifications:

  (i) Structural steel sections framing into the connections are considered to provide confinement over a width equal to that of face bearing stiffener plates welded to the beams between the flanges.

  (ii) Lap splices are permitted for perimeter ties when confinement of the splice is provided by Face Bearing Plates or other means that prevents spalling of the concrete cover in the systems described in Sections 10, 11, 12 and 15.

(c) The longitudinal bar sizes and layout in reinforced concrete and Composite Columns shall be detailed to minimize slippage of the bars through the beam-to-column connection due to high force transfer associated with the change in column moments over the height of the connection.

8. COMPOSITE PARTIALLY RESTRAINED (PR) MOMENT FRAMES (C-PRMF)

8.1. Scope

This Section is applicable to frames that consist of structural steel columns and Composite Beams that are connected with Partially Restrained (PR) moment connections that meet the requirements in LRFD Specification Section A2. C-PRMF shall be designed so that under earthquake loading yielding occurs in the ductile components of the composite PR beam-to-column moment connections. Limited yielding is permitted at other locations, such as the Column Base connection. Connection flexibility and Composite Beam action shall be accounted for in determining the dynamic characteristics, strength and drift of C-PRMF. C-PMRF shall meet the requirements of this section.

8.2. Columns

Structural steel columns shall meet the requirements in Part I Section 8 and the LRFD Specification. The effect of PR moment connections on stability of individual columns and the overall frame shall be considered in C-PRMF.

8.3. Composite Beams

Composite Beams shall meet the requirements in LRFD Specification Chapter I. For the purposes of analysis, the stiffness of beams shall be determined with an effective moment of inertia of the composite section.

8.4. Partially Restrained (PR) Moment Connections

The Required Strength for the beam-to-column PR moment connections shall be determined from the load combinations stipulated by the Applicable Building Code, including consideration of the effects of connection flexibility and second-order moments. In addition, composite connections shall have a Nominal Strength that is at least equal to 50 percent of M_p, where M_p is the nominal plastic flexural strength of the connected structural steel beam ignoring composite action. Connections shall meet the requirements in Section 7 and shall have an inelastic rotation capacity of 0.015 radians and a total rotation capacity of 0.03 radians that is substantiated by cyclic testing as described in Part I Section 9.2a.

9. COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

9.1. Scope

This Section is applicable to moment-resisting frames that consist of either composite or reinforced concrete columns and either structural steel or Composite Beams. C-SMF shall be designed assuming that under the Design Earthquake significant inelastic deformations will occur, primarily in the beams, but with limited inelastic deformations in the columns and/or connections. C-SMF shall meet the requirements of this section.

9.2. Columns

Composite Columns shall meet the requirements for special seismic systems in Sections 6.4 or 6.5. Reinforced concrete columns shall meet the requirements in ACI 318 Chapter 21, excluding Section 21.10.

9.3. Beams

Composite Beams shall meet the requirements in Section 6.3. Neither structural steel nor composite trusses are permitted as flexural members to resist seismic loads in C-SMF unless it is demonstrated by testing and analysis that the particular system provides adequate ductility and energy dissipation capacity.

9.4. Moment Connections

The Required Strength of beam-to-column moment connections shall be determined from the shear and flexure associated with the nominal plastic flexural strength of the beams framing into the connection. The nominal connection strength shall meet the requirements in Section 7. In addition, the connections shall be capable of sustaining an inelastic beam rotation of 0.03 radians. When the beam flanges are interrupted at the connection, the inelastic rotation capacity shall be demonstrated as specified in Part I Section 9 for connections in SMF. For connections to reinforced concrete columns with a beam that is continuous through the column so that welded joints are not required in the flanges and the connection is not otherwise susceptible to premature fractures, the inelastic rotation capacity shall be demonstrated by testing or other substantiating data.

9.5. Column-Beam Moment Ratio

The minimum flexural strength and design of reinforced concrete columns shall meet the requirements in ACI 318 Section 21.4.2. The minimum flexural strength and design of Composite Columns shall meet the requirements in Part I Section 9.6 with the following modifications:

(1) The flexural strength of the Composite Column M^*_{pc} shall meet the requirements in LRFD Specification Chapter I with consideration of the applied axial load, P_u.

(2) The force limit for the exceptions in Part I Section 9.6(a) shall be $P_u < 0.1P_o$.

(3) Composite Columns exempted by the minimum flexural strength requirement in Part I Section 9.6 shall have transverse reinforcement that meets the requirements in Section 6.4c(4).

10. COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)

10.1 Scope

This Section is applicable to moment resisting frames that consist of either composite or reinforced concrete columns and either structural steel or Composite Beams. C-IMF shall be designed assuming that under the Design Earthquake inelastic deformation will occur primarily in the beams but with moderate inelastic deformation in the columns and/or connections. C-IMF shall meet the requirements of this section.

10.2. Columns

Composite Columns shall meet the requirements for intermediate seismic systems in Section 6.4 or 6.5. Reinforced concrete columns shall meet the requirements in ACI 318 Section 21.10.

10.3. Beams

Structural steel and Composite Beams shall meet the requirements in the LRFD Specification.

10.4. Moment Connections

The nominal connection strength shall meet the requirements in Section 7. The Required Strength of beam-to-column connections shall meet one of the following requirements:

(a) The connection Design Strength shall meet or exceed the forces associated with plastic hinging of the beams adjacent to the connection.

(b) The connection Design Strength shall meet or exceed the Required Strength generated by load combinations stipulated by the Applicable Building Code, including the Amplified Seismic Load.

(c) The connections shall demonstrate an inelastic rotation capacity of at least 0.02 radians in cyclic tests.

11. COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)

11.1. Scope

This Section is applicable to moment resisting frames that consist of either composite or reinforced concrete columns and structural steel or Composite Beams. C-OMF shall be designed assuming that under the Design Earthquake limited inelastic action will occur in the beams, columns and/or connections. C-OMF shall meet the requirements of this section.

11.2. Columns

Composite Columns shall meet the requirements for ordinary seismic systems in Section 6.4 or 6.5. Reinforced concrete columns shall meet the requirements in ACI 318, excluding Chapters 21.

11.3. Beams

Structural steel and Composite Beams shall meet the requirements in the LRFD Specification.

11.4. Moment Connections

Connections shall be designed for the applied factored load combinations and their Design Strength shall meet the requirements in Section 7.

12. COMPOSITE ORDINARY BRACED FRAMES (C-OBF)

12.1. Scope

This Section is applicable to concentrically and Eccentrically Braced Frame systems that consist of either composite or reinforced concrete columns, structural steel or Composite Beams, and structural steel or Composite Braces. C-OBF shall be designed assuming that under the Design Earthquake limited inelastic action will occur in the beams, columns, braces, and/or connections. C-OBF shall meet the requirements of this section.

12.2. Columns

Reinforced-Concrete-Encased Composite Columns shall meet the requirements for ordinary seismic systems in Sections 6.4. Concrete-Filled Composite Columns shall meet the requirements in Section 6.5. Reinforced concrete columns shall meet the requirements in ACI 318 excluding Chapter 21.

12.3. Beams

Structural steel and Composite Beams shall meet the requirements in the LRFD Specification.

12.4. Braces

Structural steel braces shall meet the requirements in the LRFD Specification. Composite Braces shall meet the requirements for Composite Columns in Section 12.2.

12.5. Connections

Connections shall be designed for the applied load combinations stipulated by the Applicable Building Code and their Design Strength shall meet the requirements in Section 7.

13. COMPOSITE CONCENTRICALLY BRACED FRAMES (C-CBF)

13.1. Scope

This Section is applicable to braced systems that consist of concentrically connected members. Minor eccentricities are permitted if they are accounted for in the design. Columns shall be either composite structural steel or reinforced concrete. Beams and braces shall be either structural steel or composite structural steel. C-CBF shall be designed so that under the loading of the Design Earthquake inelastic action will occur primarily through tension yielding and/or buckling of braces. C-CBF shall meet the requirements of this section.

13.2. Columns

Structural steel columns shall meet the requirements in Part I Section 8. Composite structural steel columns shall meet the requirements for special systems in Section 6.4 or 6.5. Reinforced concrete columns shall meet the requirements for structural truss elements in ACI 318 Chapter 21.

13.3. Beams

Structural steel and Composite Beams shall meet the requirements in the LRFD Specification.

13.4. Braces

Structural steel braces shall meet the requirements for SCBF in Part I Section 13. Composite Braces shall meet the requirements for Composite Columns in Section 13.2.

13.5. Bracing Connections

Bracing connections shall meet the requirements in Section 7 and Part I Section 13.

14. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

14.1. Scope

This Section is applicable to braced systems for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and column or intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace. C-EBF shall be designed so that inelastic deformations will occur only as shear yielding in the Links. The diagonal braces, columns, and beam segments outside of the Link shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened Link. Columns shall be either composite or reinforced concrete. Braces shall be structural steel. Links shall be structural steel as described in this Section. The Design Strength of members shall meet the requirements in the LRFD Specification, except as modified in this Section. C-EBF shall meet the requirements in Part I Section 15, except as modified in this Section.

14.2. Columns

Reinforced concrete columns shall meet the requirements for structural truss elements in ACI 318 Chapter 21. Composite Columns shall meet the requirements for special seismic systems in Sections 6.4 or 6.5. Additionally, where a Link is adjacent to a reinforced concrete column or reinforced-concrete-encased column, transverse reinforcement meeting the requirements in ACI 318 Section 21.4.4 (or Section 6.4c.6.a for Composite Columns) shall be provided above and below the Link connection.

All columns shall meet the requirements in Part I Section 15.8.

14.3. Links

Links shall be unencased structural steel and shall meet the requirement for EBF Links in Part I Section 15. It is permitted to encase the portion of the beam outside of the Link with reinforced concrete. Beams containing the Link are permitted to act compositely with the floor slab using shear connectors along all or any portion of the beam if the composite action is considered when determining the Nominal Strength of the Link.

14.4. Braces

Structural steel braces shall meet the requirements for EBF in Part I Section 15.

14.5. Connections

In addition to the requirements for EBF in Part I Section 15, connections shall meet the requirements in Section 7.

15. ORDINARY REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-ORCW)

15.1. Scope

The requirements in this Section apply when reinforced concrete walls are composite with structural steel elements, either as infill panels, such as reinforced concrete walls in structural steel frames with unencased or reinforced-concrete-encased structural steel sections that act as Boundary Members, or as structural steel Coupling Beams that connect two adjacent reinforced concrete walls. Reinforced concrete walls shall meet the requirements in ACI 318 excluding Chapter 21. C-ORCW shall meet the requirements of this section.

15.2. Boundary Members

When unencased structural steel sections function as Boundary Members in reinforced concrete infill panels, the structural steel sections shall meet the requirements in the LRFD Specification. The required axial strength of the Boundary Member shall be determined assuming that the shear forces are carried by the reinforced concrete wall and the entire gravity and overturning forces are carried by the Boundary Members in conjunction with the shear wall. The reinforced concrete wall shall meet the requirements in ACI 318 excluding Chapter 21.

When fully reinforced-concrete-encased structural steel sections function as Boundary Members in reinforced concrete infill panels, the analysis shall be based upon a transformed concrete section using elastic material properties. The wall shall meet the requirements in ACI 318 excluding Chapter 21. When the reinforced-concrete-encased structural steel Boundary Member qualifies as a Composite Column as defined in LRFD Specification Chapter I, it shall be designed to meet the ordinary seismic system requirements in Section 6.4. Otherwise, it shall be designed as a Composite Column to meet the requirements in ACI 318.

Headed shear studs or welded reinforcement anchors shall be provided to transfer vertical shear forces between the structural steel and reinforced concrete. Headed shear studs, if used, shall meet the requirements in LRFD Specification Chapter I. Welded reinforcement anchors, if used, shall meet the requirements in AWS D1.4.

15.3. Coupling Beams

Structural steel Coupling Beams that are used between two adjacent reinforced concrete walls shall meet the requirements in the LRFD Specification and this Section:

Coupling Beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the maximum possible combination of moment and shear that can be generated by the nominal bending and shear strength of the Coupling Beam. The embedment length shall be considered to begin inside the first layer of confining reinforcement in the wall Boundary Member. Connection strength for the transfer of loads between the Coupling Beam and the wall shall meet the requirements in Section 7.

Vertical wall reinforcement with design axial strength equal to the nominal shear strength of the Coupling Beam shall be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement shall extend a distance of at least one tension development length above and below the flanges of the Coupling Beam. It is permitted to use vertical reinforcement placed for other purposes, such as for vertical Boundary Members, as part of the required vertical reinforcement.

16. SPECIAL REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-SRCW)

16.1. Scope

C-SRCW systems shall meet the requirements in Section 15 for C-ORCW and the shear-wall requirement in ACI 318 including Chapter 21, except as modified in this Section.

16.2. Boundary Members

In addition to the requirements in Section 15.2a, unencased structural steel columns shall meet the requirements in Part I Sections 5, 6 and 8.

Walls with reinforced-concrete-encased structural steel Boundary Members shall meet the requirements in Section 15.2 as wells as the requirements in this Section. The wall shall meet the requirements in ACI 318 including Chapter 21. Reinforced-concrete-encased structural steel Boundary Members that qualify as Composite Columns in LRFD Specification Chapter I shall meet the special seismic system requirements in Section 6.4. Otherwise, such members shall be designed as composite compression members to meet the requirements in ACI 318 including the special seismic requirements for Boundary Members in Chapter 21. Transverse reinforcement for confinement of the composite Boundary Member shall extend a distance of $2h$ into the wall where h is the overall depth of the Boundary Member in the plane of the wall.

Headed shear studs or welded reinforcing bar anchors shall be provided as specified in Section 15.2c. For connection to unencased structural steel sections, the Nominal Strength of welded reinforcing bar anchors shall be reduced by 25 percent from their Static Yield Strength.

16.3. Coupling Beams

In addition to the requirements in Section 15.3a, structural steel Coupling Beams shall meet the requirements in Part I Sections 15.2 and 15.3. When required in Part I Section 15.3, the coupling rotation shall be assumed as 0.08 radians unless a smaller value is justified by rational analysis of the inelastic deformations that are expected under the Design Earthquake. Face Bearing Plates shall be provided on both sides of the Coupling Beams at the face of the reinforced concrete wall. These stiffeners shall meet the detailing requirements in Part I Section 15.3.

Vertical wall reinforcement as specified in Section 15.3 shall be confined by transverse reinforcement that meets the requirements for Boundary Members in ACI 318 Section 21. 7.2.

17. COMPOSITE STEEL PLATE SHEAR WALLS (C-SPW)

17.1. Scope

This Section is applicable to structural walls consisting of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite Boundary Members. C-SPW shall meet the requirements of this section.

17.2. Wall Elements

17.2a. Nominal Shear Strength

The nominal shear strength of C-SPW with a stiffened plate conforming to Section 17.2b shall be determined as:

$$V_{ns} = 0.6 A_{sp} F_y \tag{17-1}$$

where

V_{ns} = nominal shear strength of the steel plate, kips (N)
A_{sp} = horizontal area of stiffened steel plate, in.^{2} (mm^2)
F_y = specified minimum yield strength of the plate, ksi (MPa)

The nominal shear strength of C-SPW with a plate that does not meet the stiffening requirements in Section 17.2b shall be based upon the strength of the plate, excluding the strength of the reinforced concrete, and meet the requirements in the LRFD Specification, including the effects of buckling of the plate.

17.2b. Detailing Requirements

The steel plate shall be adequately stiffened by encasement or attachment to the reinforced concrete if it can be demonstrated with an elastic plate buckling analysis that the composite wall can resist a nominal shear force equal to V_{ns}. The concrete thickness shall be a minimum of 4 in. (102 mm) on each side when concrete is provided on both sides of the steel plate and 8 in. (203 mm) when concrete is provided on one side of the steel plate. Headed shear stud connectors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet the detailing requirements in ACI 318 Section 14.3. The reinforcement ratio in both directions shall not be less than 0.0025; the maximum spacing between bars shall not exceed 18 in. (457 mm).

The steel plate shall be continuously connected on all edges to structural steel framing and Boundary Members with welds and/or slip-critical high-strength bolts to develop the nominal shear strength of the plate. The Design Strength of welded and bolted connectors shall meet the additional requirements in Part I Section 7.

17.3. Boundary Members

Structural steel and composite Boundary Members shall be designed to meet the requirements in Section 16.2.

Boundary Members shall be provided around openings as required by analysis.

PART III. ALLOWABLE STRESS DESIGN (ASD) ALTERNATIVE

As an alternative to the Load and Resistance Factor Design (LRFD) provisions for structural steel design in Part I, the use of the Allowable Stress Design (ASD) provisions in this Part is permitted. All requirements of Part I shall be met except as modified or supplemented in this Part. When using this Part, the terms "LRFD Specification", "FR" and "PR" in Part I shall be taken as "ASD Specification", "Type 1", and "Type 3", respectively.

1. SCOPE

Substitute the following for PART I Section 1 in its entirety:

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 *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* including Supplement No. 1, hereinafter referred to as the ASD Specification. All members and connections in the Seismic Force Resisting System shall be proportioned as required in the ASD Specification to resist the applicable load combinations and shall meet the requirements in these Provisions.

Part III includes the Part I Glossary and Appendix S.

2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

Substitute the following for the first two paragraphs of Part I Section 2:

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

American Institute of Steel Construction
Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design, June 1, 1989 including Supplement No. 1, December 17, 2001

Substitute the following for the last paragraph of Part I Section 2:

Research Council on Structural Connections
Specification for Structural Joints Using ASTM A325 or A490 Bolts, June 23, 2000, Appendix B

4. LOADS, LOAD COMBINATIONS, AND NOMINAL STRENGTHS

Substitute the following for Part I Section 4.2 in its entirety:

4.2 Nominal Strength

The Nominal Strengths of members and connections shall be determined as follows:

Replace ASD Specification Section A5.2 with the following: "The Nominal Strength of structural steel members and connections for resisting seismic forces acting alone or in combination with dead and live loads shall be determined by multiplying 1.7 times the allowable stresses in Section D, E, F, G, H, J, and K."

Amend the first paragraph of ASD Specification Section N1 by deleting "or earthquake" and adding: "The Nominal Strength of members and connections shall be determined by the requirements contained herein. Except as modified in these provisions, all pertinent requirements of Chapters A through M shall govern."

In ASD Specification Section H1 the definition of F_e' shall read as follows:

$$F_e' = \frac{\pi^2 E_s}{(Kl_b / r_b)^2} \tag{4-1}$$

where:

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

4.3. Design Strength

The Design Strength of structural steel members and connections subjected to seismic forces in combination with other prescribed loads shall be determined by converting allowable stresses into Nominal Strengths and multiplying such Nominal Strengths by the Resistance Factors given in Table III-4-1.

TABLE III-4-1
Resistance Factors for ASD

Limit State	Resistance Factor
Tension	
Yielding	0.90
Rupture	0.75
Compression buckling	0.85
Flexure	
Yielding	0.90
Rupture	0.75
Shear	
Yielding	0.90
Rupture	0.75
Torsion	
Yielding	0.90
Buckling	0.90
Complete-joint-penetration groove welds	
Tension or compression normal to effective area	Base metal 0.90 Weld metal 0.90
Shear on effective area	Base metal 0.90 Weld metal 0.80
Partial-joint-penetration groove welds	
Compression normal to effective area	Base metal 0.90 Weld metal 0.90
Tension normal to effective area	Base metal 0.90 Weld metal 0.80
Shear parallel to axis of weld	Weld metal 0.75
Fillet welds	
Shear on effective area	Weld metal 0.75
Plug or slot welds	
Shear parallel to faying surface (on effective area)	Weld metal 0.75
Bolts	
Tension rupture, shear rupture, combined tension and shear	0.75
Slip resistance for bolts in standard holes, oversized holes, and short-slotted holes	1.0
Slip resistance for bolts in long-slotted holes with the slot perpendicular to the direction of the slot	1.0
Slip resistance for bolts in long-slotted holes with the slot parallel to the direction of the slot	0.85
Connecting elements	0.90

Tension yielding, shear yielding	0.75
Bearing strength at bolt holes, tension rupture, shear rupture, block shear rupture	Bearing on steel 0.75 Bearing on concrete 0.60
Contact bearing	
Flanges and webs with concentrated forces	0.90
Local flange bending, compression buckling of web	1.0
Local web yielding	0.75
Web crippling, Panel Zone web shear	0.85
Sidesway web buckling	

7. CONNECTIONS, JOINTS, AND FASTENERS

7.2. Bolted Joints

Substitute the following for Part I Section 7.2 fourth paragraph in its entirety:

The design resistance to shear and combined tension and shear of bolted joints shall be determined in accordance with the ASD Specification Sections J3.5 and J3.7, except that the allowable bearing stress at bolt holes F_p shall not be taken greater than $1.2F_u$.

9. SPECIAL MOMENT FRAMES

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

Substitute the following for Part I Section 9.3a in its entirety:

The required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested connection. As a minimum, the required shear strength R_u of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength $\phi_v R_v$ of the panel zone shall be determined using $\phi_v = 1.0$.

When $P_u \leq 0.75P_y$,

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

When $P_u > 0.75P_y$,

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right]\left[1.9 - \frac{1.2P_u}{P_y}\right]. \quad (9\text{-}1a)$$

where

t_p = total thickness of Panel Zone including doubler plate(s), in. (mm)
d_c = overall column depth, in. (mm)
b_{cf} = width of the column flange, in. (mm)
t_{cf} = thickness of the column flange, in. (mm)
d_b = overall beam depth, in. (mm)
F_y = specified minimum yield strength of the Panel Zone steel, ksi (MPa)

9.7 Beam-to-Column Connection Restraint

Substitute the following for Part I Section 9.7b(1) in its entirety:

The required column strength shall be determined from the ASD load combinations stipulated in the Applicable Building Code, except that E shall be taken as the lesser of:

(a) The Amplified Seismic Load

(b) 125 percent of the frame Design Strength based upon either the beam design flexural strength or Panel Zone design shear strength

12. SPECIAL TRUSS MOMENT FRAMES

12.4 Nominal Strength of Non-special Segment Members

Substitute the following for the first sentence in Part I Section 12.4:

Members and connections of STMF, except those in the special segment defined in Section 12.2, shall have a Design Strength to resist ASD load combinations as stipulated by the Applicable Building Code replacing the earthquake load term E with the lateral loads necessary to develop the expected vertical nominal shear strength in the special segment V_{ne} given as: [balance to remain unchanged]

12.6 Lateral Bracing

Substitute the following for the first sentence in Part I Section 12.6:

The top and bottom chords of the trusses shall be laterally braced at the ends of the special segment, and at intervals not to exceed L_c according to ASD Specification Section F1, along the entire length of the truss.

13. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

Substitute the following for Part I Section 13.4a(2) in its entirety:

(2) A beam that is intersected by braces shall be designed to support the effects of all tributary dead and live loads assuming that the bracing is not present.

Substitute the following for Part I Section 13.4a(3) in its entirety:

(3) A beam that is intersected by braces shall be designed to resist the effects of ASD load combinations as stipulated by the Applicable Building Code, 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.

14. ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

Substitute the following for Part I Section 14.2 in its entirety:

14.2. Strength

The Required Strength of the members and connections, other than brace connections, in OCBFs shall be determined using the ASD load combinations stipulated by the Applicable Building Code except E shall be taken as the Amplified Seismic Load. The Design Strength of brace connections shall equal or exceed the expected tensile strength of the brace, determined as $R_y F_y A_g$. Braces with Kl/r greater than $4.23\sqrt{E_s / F_y}$ shall not be used in V or inverted-V configurations.