Seismic Provisions for Structural Steel Buildings

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PREFACE

This Preface is not a part of ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings,* but is included for informational purposes only.

The AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-10) is intended to cover common design criteria. Accordingly, it is not feasible for it to also cover all of the special and unique problems encountered within the full range of structural design practice. This document, the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-10) (hereafter referred to as the *Provisions*) is a separate consensus standard that addresses one such topic: the design and construction of structural steel and composite structural steel/reinforced concrete building systems for high-seismic applications.

A list of Symbols, and Glossary are part of this document. Terms that appear in the Glossary are generally italicized where they first appear in a sub-section, throughout these *Provisions*. A nonmandatory Commentary with background information is also provided.

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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TABLE OF CONTENTS

SYMBOLS

The symbols listed below shall be used in addition to or replacements for those in the AISC *Specification for Structural Steel Buildings*. Where there is a duplication of the use of a symbol between this Specification and the AISC *Specification for Structural Steel Buildings*, the symbol listed herein shall take precedence. The section or table number in the right-hand column refers to where the symbol is first used.

Symbol	Definition	Reference *(to be provided in final)*
A_b	Cross-sectional area of a horizontal boundary element (HBE), in.^{2} (mm^2)	
A_c	Cross-sectional area of a vertical boundary element (VBE), in.^{2} (mm^2)	
A_f	Flange area, in.^{2} (mm^2)	
A_g	Gross area, in.^{2} (mm^2)	
A_{lw}	Link web area (excluding flanges), in.^{2} (mm^2)	
A_s	Cross-sectional area of the structural steel core, in.^{2} (mm^2)	
A_s	Area of transfer reinforcement in coupling beam, in.^{2} (mm^2)	
A_{sc}	Area of the yielding segment of steel core, in.^{2} (mm^2)	
A_{sh}	Minimum area of tie reinforcement, in.^{2} (mm^2)	
A_{sp}	Horizontal area of stiffened steel plate in composite steel plate shear wall, in.^{2} (mm^2)	
A_{st}	Horizontal cross-sectional area of *link* stiffener, in.^{2} (mm^2)	
A_{tb}	Area of transfer reinforcement in coupling beam, in.^{2} (mm^2)	
C_a	Ratio of required strength to available strength	
C_d	Coefficient relating relative brace stiffness and curvature	
D	Dead load due to the weight of the structural elements and permanent features on the building, kips (N)	
D	Outside diameter of round HSS, in. (mm)	
E	Earthquake load	
E	Modulus of elasticity of steel, E = 29,000 ksi (200 000 MPa)	
F_y	Specified minimum yield stress of the type of steel to be used, ksi (MPa). As used in the *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).	

F_{yb} F_y of a beam, ksi (MPa) ....

F_{yc} F_y of a column, ksi (MPa) ....

F_{yh} Specified minimum yield stress of the ties, ksi (MPa) ....

F_{ysc} Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, ksi (MPa) ....

F_u Specified minimum tensile strength, ksi (MPa) ....

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) ....

H_c Clear height of story, which may be taken as the distance between the bottom of the flange of the beam above and either the top of the flange of the beam below, or the top of the structural slab, if present, in. (mm) ....

I Moment of inertia, in.^{4} (mm^4) ....

I_c Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, in.^{4} (mm^4) ....

K Effective length factor for prismatic member ....

L Live load due to occupancy and moveable equipment, kips (kN) ....

L Span length of the truss, in. (mm) ....

L Distance between VBE centerlines, in. (mm) ....

L_b Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)...

L_{cf} Clear distance between column flanges, in. (mm) ....

L_e Embedment length of coupling beam, in. (mm)........................

L_h Distance between plastic hinge locations, in. (mm) ....

L_p Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, in. (mm) ....

L_{pd} Limiting laterally unbraced length for plastic analysis, in. (mm) ....

L_s Length of the *special segment*, in. (mm) ....

M_a Required flexural strength, using ASD load combinations, kip-in. (N-mm) ....

M_{av} Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on ASD load combinations, kip-in. (N-mm)...

M_n Nominal flexural strength, kip-in. (N-mm) ....

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

$M_{n,PR}$ Nominal flexural strength of PR connection at a rotation of 0.02 rad, kip-in. (N-mm)

M_p Nominal plastic flexural strength, kip-in. (N-mm) ....

M_{pa} Nominal plastic flexural strength modified by axial load, kip-in. (N-mm) ....

M_{pb} Nominal plastic flexural strength of the beam, kip-in. (N-mm) ....

M_{pc} Nominal plastic flexural strength of the column, kip-in. (N-mm) ....

M^{*}_{pcc} Nominal flexural strength of composite or reinforced concrete column with consideration of the required axial strength, kip-in. (N-mm)

$M_{p,exp}$ Expected plastic moment, kip-in. (N-mm) ....

$M^{*}_{p,exp}$ Expected flexural strength at the intersection of the beam and column centerlines, kip-in. (N-mm) ....

M_r Expected flexural strength, kip-in. (N-mm) ....

M_{uv} Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD load combinations, kip-in. (N-mm) ....

M_u Required flexural strength, using LRFD load combinations, kip-in. (N-mm) ....

$M_{u,exp}$ Expected required flexural strength, kip-in. (N-mm) ....

M_{uv} Moment due to shear amplification from the location of the plastic hinge to the column centerline, kip-in. (N-mm) ....

P_a Required axial strength of a column using ASD load combinations, kips (N) ....

P_{ac} Required compressive strength of columns using ASD load combinations, kips (N) ....

P_b Required strength of lateral brace at ends of the link, kips (N) ....

P_b Axial design strength of wall at balanced condition, kips (N) ..

P_c Available axial strength of a column, kips (N) ....

P_n Nominal axial strength of a column, kips (N) ....

P_n Nominal compressive strength of the composite column calculated in accordance with the *Specification*, kips (N)

P_{nc} Nominal axial compressive strength of diagonal members of the special segment, kips (N) ....

P_{nt} Nominal axial tensile strength of diagonal members of the special segment, kips (N) ....

P_o Nominal axial strength of a composite column at zero eccentricity, kips (N) ....

P_r Required compressive strength, kips (N) ....

P_{rc} Required compressive strength of columns using ASD or LRFD load combinations, kips (N) ....

P_u Required axial strength using LRFD load combinations, kips (N) ....

Required axial strength of a composite column, kips (N) ....

P_{uc} Required compressive strength of columns using LRFD load combinations, kips (N) ....

P_y Nominal axial yield strength of a member, equal to F_yA_g, kips

(N) ....
P_{ysc} Axial yield strength of steel core, kips (N) ....
Q_1 Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link..............................
R Seismic response modification coefficient ....
R_n Nominal strength, kips (N) ....
R_t Ratio of the expected tensile strength to the specified minimum tensile strength F_u, as related to overstrength in material yield stress R_y ....
R_u Required strength ....
R_v Panel zone nominal shear strength ....
R_y Ratio of the expected yield stress to the specified minimum yield stress, F_y ....
S Snow load, kips (N) ....
V_a Required shear strength using ASD load combinations, kips (N) ....
V_{comp} Nominal shear strength of composite coupling beam, kips (N)..
V_n Nominal shear strength of a member, kips (N) ....
$V_{n,comp}$ Nominal shear strength of composite coupling beam, kips (N)
V_{ne} Expected vertical shear strength of the special segment, kips (N) ....
V_{ns} Nominal shear strength of the steel plate in a composite plate shear walls, kips (N) ....
V_p Nominal shear strength of an active link, kips (N) ....
V_{pa} Nominal shear strength of an active link modified by the axial load magnitude, kips (N) I
V_u Required shear strength using LRFD load combinations, kips (N) ....
Y_{con} Distance from top of steel beam to top of concrete slab or encasement, in. (mm) ....
Y_{PNA} Maximum distance from the maximum concrete compression fiber to the plastic neutral axis, in. (mm)...
Z Plastic section modulus of a member, in.^{3} (mm^3) ....
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_x Plastic section modulus x-axis, in.^{3} (mm^3)......................................
Z_{RBS} Minimum plastic section modulus at the reduced beam section, in.^{3} (mm^3) ....
b Width of compression element as defined in *Specification* Section B4.1, in. (mm) ....
b_{cf} Width of column flange, in. (mm) ....
b_f Flange width, in. (mm) ....
b_w Width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear, in. (mm) ....

b_w Wall width, in. (mm) ....
d Nominal fastener diameter, in. (mm) ....
d Overall beam depth, in. (mm) ....
d_c Overall column depth, in. (mm) ....
d_z Overall panel zone depth between continuity plates, in. (mm) ....
e EBF link length, in. (mm) ....
f'_c Specified compressive strength of concrete, ksi (MPa) ....
h Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; and for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, in. (mm) ....
h Distance between horizontal boundary element centerlines, in. (mm) ....
h_{cc} Cross-sectional dimension of the confined core region in composite columns measured center-to-center of the transverse reinforcement, in. (mm) ....
h_o Distance between flange centroids, in. (mm) ....
l Unbraced length between stitches of built-up braces, in. (mm)
l Unbraced length of compression or bracing member, in. (mm) ....
r Governing radius of gyration, in. (mm) ....
r_y Radius of gyration about y axis, in. (mm)..
s Spacing of transverse reinforcement measured along the longitudinal axis of the structural composite member, in. (mm) ....
t Thickness of connected part, in. (mm) ....
t Thickness of element, in. (mm) ....
t Thickness of column web or doubler plate, in. (mm) ....
t_{bf} Thickness of beam flange, in. (mm) ....
t_{cf} Thickness of column flange, in. (mm)...
t_f Thickness of flange, in. (mm) ....
t_p Thickness of panel zone including doubler plates, in. (mm) ....
t_w Thickness of web, in. (mm) ....
w_z Width of panel zone between column flanges, in. (mm) ....
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 ....
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 ....
ΣM^*_{pcc} Sum of the nominal flexural strengths of composite columns above and below the joint at the intersection of the beam and column centerlines ....

$\Sigma M^*_{p,exp}$ Sum of the expected flexural strengths of steel beams or concrete-encased composite beams at the intersection of the beam and column centerlines ....

β Compression strength adjustment factor ....
Δ Design story drift ....
Δ_b Deformation quantity used to control loading of test specimen (total brace end rotation for the subassemblage test specimen; total brace axial deformation for the brace test specimen) ....
Δ_{bm} Value of deformation quantity, Δ_b, corresponding to the design story drift ....
Δ_{by} Value of deformation quantity, Δ_b, at first significant yield of test specimen ....
Ω Safety factor ....
Ω_b Safety factor for flexure = 1.67 ....
Ω_c Safety factor for compression = 1.67 ....
Ω_o Horizontal seismic overstrength factor ....
Ω_v Safety factor for shear strength of panel zone of beam-to-column connections ....
α Angle of diagonal members with the horizontal ....
α Angle of web yielding in radians, as measured relative to the vertical ....
δ Deformation quantity used to control loading of test specimen ....
δ_y Value of deformation quantity δ at first significant yield of test specimen ....
ρ' Ratio of stress due to required axial strength (P_r / A_g) to required stress due to required shear strength (V_r/A_w) of a link ....
λ_{hd}, λ_{md} Limiting slenderness parameter for highly and moderately ductile compression elements, respectively ....
ϕ Resistance factor ....
ϕ_b Resistance factor for flexure ....
ϕ_c Resistance factor for compression ....
ϕ_v Resistance factor for shear ....
θ Interstory drift angle, radians ....
γ_{total} Link rotation angle ....
ω Strain hardening adjustment factor ....

Glossary

The terms listed below shall be used in addition to those in the AISC *Specification for Structural Steel Buildings*. Some commonly used terms are repeated here for convenience. Glossary terms are generally *italicized* throughout these *Provisions* and Commentary, where they first appear within a section.

Notes:

(1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards developers.

(2) Terms designated with * are usually qualified by the type of *load effect,* for example, *nominal tensile strength*, *available compressive strength*, *design flexural strength*.

(3) Terms designated with ** are usually qualified by the type of component, for example, web *local buckling*, flange *local bending*.

Adjusted brace strength. Strength of a brace in a *buckling-restrained braced frame* at deformations corresponding to 2.0 times the *design story drift*.

Allowable strength†. Nominal strength divided by the safety factor, R_n / Ω.

Applicable building code†. Building code under which the structure is designed.

Amplified seismic load. Horizontal component of earthquake load E multiplied by Ω_o, where E and the horizontal component of E are specified in the *applicable building code*.

Authority having jurisdiction. Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this Standard.

Available strength†. *Design strength* or *allowable strength*, as appropriate.

ASD (Allowable Strength Design)†. Method of proportioning structural components such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the ASD load combinations.

ASD load combination†. Load combination in the *applicable building code* intended for *allowable strength design* (allowable stress design).

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

Braced frame†. An essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.

Brace test specimen. A single buckling-restrained brace element used for laboratory testing intended to model the brace in the *prototype*.

Buckling-restrained brace. A pre-fabricated, or manufactured, brace element consisting of a steel core and a buckling-restraining system as described in Section F4 and qualified by testing as required in Section J3.

Buckling-restrained braced frame (BRBF). A diagonally braced frame employing buckling-restrained braces and meeting the requirements of Section F4.

Buckling-restraining system. System of restraints that limits buckling of the steel core in BRBF. This system includes the casing surrounding the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the *design story drift*.

Cantilever column system (CCS). A seismic force resisting system in which the seismic forces are resisted by one or more columns that are cantilevered from the foundation, or from the diaphragm level below.

Casing. Element that resists forces transverse to the axis of the diagonal brace thereby restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force along the axis of the diagonal brace.

Collector. Also known as drag strut, member that serves to transfer loads between floor diaphragms and the members of the *seismic force resisting system*.

Column base. Assemblage of structural shapes, plates, connectors, bolts and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

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

Composite beam. Structural steel beam in contact with and acting compositely with a reinforced concrete slab designed to act compositely for seismic forces.

Composite brace. Reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a diagonal brace.

Composite column. Reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a column.

Composite eccentrically braced frame (C-EBF). Composite braced frame meeting the requirements of Section H3.

Composite intermediate moment frame (C-IMF). Composite moment frame meeting the requirements of Section G2.

Composite ordinary braced frame (C-OBF). Composite braced frame meeting the requirements of Section H1.

Composite ordinary moment frame (C-OMF). Composite moment frame meeting the requirements of Section G1.

Composite ordinary reinforced concrete shear wall with steel elements (C-ORCW). Composite shear walls meeting the requirements of Section H4.

Composite partially-restrained moment frame (C-PRMF). Composite moment frame meeting the requirements of Section G4.

Composite shear wall. Reinforced concrete wall that has unencased or reinforced-concrete-encased structural steel sections as *boundary members*.

Composite slab. Reinforced concrete slab supported on and bonded to a formed steel deck that acts as a diaphragm to transfer load to and between elements of the *seismic force resisting system*.

Composite special concentrically braced frame (C-SCBF). Composite braced frame meeting the requirements of Section H2.

Composite special moment frame (C-SMF). Composite moment frame meeting the requirements of Section G3.

Composite special reinforced concrete shear wall with steel elements (C-SRCW).Composite shear walls meeting the requirements of Section H5.

Composite steel plate shear wall (C-SPW). Wall consisting of steel plate with reinforced concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel plate and meeting the requirements of Section H6.

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

Contractor. Fabricator or erector, as applicable.

Coupling beam. Structural steel or composite beam connecting adjacent reinforced concrete wall elements so that they act together to resist lateral loads.

Demand critical weld. Weld so designated by these *Provisions*.

Design earthquake. The earthquake represented by the *design response spectrum* as specified in the *applicable building code*.

Design methodology. A set of step-by-step procedures, based on calculation or experiment, used to determine sizes, lengths, and details in the design of buckling-restrained braces and their connections.

Design story drift. Calculated story drift, including the effect of expected inelastic action, due to design level earthquake forces as determined by the *applicable building code*.

*Design strength**†. *Resistance factor* multiplied by the *nominal strength*, ϕR_n.

Diagonal brace. Inclined structural member carrying primarily axial force in a braced frame.

Dual system. 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 frames (SMF, IMF or OMF) that are capable of resisting at least 25% 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). Diagonally braced frame meeting the requirements of Section F3 that has at least one end of each diagonal brace connected to a beam with a defined eccentricity from another beam-to-brace connection or a beam-to-column connection.

Encased composite beam. Composite beam completely enclosed in reinforced concrete.

Encased composite column. Structural steel column completely encased in reinforced concrete.

Engineer of record. Licensed professional responsible for sealing the contract documents.

Exempted column. Column not meeting the requirements of Equation E3-1 for SMF.

Expected yield strength. Yield strength in tension of a member, equal to the expected yield stress multiplied by A_g.

Expected tensile strength *. Tensile strength of a member, equal to the specified minimum tensile strength, F_u, multiplied by R_t.

Expected yield stress. Yield stress of the material, equal to the specified minimum yield stress, F_y, multiplied by R_y.

Face bearing plates. Stiffeners 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 loads to the concrete through direct bearing.

Filled composite column. HSS filled with structural concrete.

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

Highly ductile member. A member expected to undergo significant plastic rotation (more than 0.02 rad) under the *design earthquake.*

Inelastic deformation. The plastic portion of deformation imposed on an element, such as plastic axial deformation in a buckling-restrained brace

Horizontal boundary element (HBE). A beam with a connection to one or more web plates in a SPSW.

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.

Intermediate boundary element (IBE). A member, other than a beam or column, that provides resistance to web plate tension adjacent to an opening in a SPSW.

Intermediate moment frame (IMF). Moment frame system that meets the requirements of Section E2.

Intermediate seismic systems. Seismic systems designed assuming moderate inelastic action occurs in some members under the design earthquake.

Interstory drift angle. Interstory displacement divided by story height, radians.

Inverted-V-braced frame. See *V-braced frame*.

k-area. The *k-area* is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC "*k*" dimension) a distance of 1½ in. (38 mm) into the web beyond the "*k*" dimension.

K-braced frame. A braced-frame configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.

Link. In EBF, the segment of a beam that is located between the ends of the connections 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. 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. Lesser of the available shear strength of the *link* based on the flexural or shear strength of the link member.

Load-carrying reinforcement. Reinforcement in composite members designed and detailed to resist the required loads.

Lowest Anticipated Service Temperature (LAST). The lowest 1-hour average temperature with a 100-year mean recurrence interval.

LRFD (Load and Resistance Factor Design) †. Method of proportioning structural components such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations*.

LRFD Load Combination†. Load combination in the *applicable building code* intended for strength design (*load and resistance factor design).*

Material test plate. A test specimen from which steel samples or weld metal samples are machined for subsequent testing to determine mechanical properties.

Measured flexural resistance. Bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Section J2.

Member brace. Member that provides stiffness and strength to control movement of another member out-of-the plane of the frame at the braced points.

Moderately ductile member A member expected to undergo moderate plastic rotation (0.02 rad or less) under the *design earthquake.*

Nominal load†. Magnitude of the *load* specified by the *applicable building code*.

*Nominal strength**†. Strength of a structure or component (without the *resistance factor* or *safety factor* applied) to resist load effects, as determined in accordance with the *Specification.*

Occupancy Category. Classification assigned to a structure based on its use as specified by the *applicable building code.*

Ordinary concentrically braced frame (OCBF). Diagonally braced frame meeting the requirements of Section F1 in which all members of the braced-frame system are subjected primarily to axial forces.

Ordinary moment frame (OMF). Moment frame system that meets the requirements of Section E1.

Ordinary seismic systems. Seismic systems designed assuming limited (less than moderate) inelastic action occurs in some members under the design earthquake.

Overstrength factor, Ω_o. Factor specified by the *applicable building code* in order to determine the amplified seismic load, where required by these *Provisions*.

Partially composite beam. Unencased composite beam with a nominal flexural strength controlled by the strength of the shear stud connectors.

Partially-restrained composite connection. Partially restrained (PR) connections as defined in the *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.

Plastic Hinge. Yielded zone that forms in a structural member when the plastic moment is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the plastic moment.

Prequalified connection. Connection that complies with the requirements of Section J1 or ANSI/AISC 358.

Protected zone. Area of members or connections of members in which limitations apply to fabrication and attachments.

Prototype. The tested connection or diagonal brace design that is to be used in the building (SMF, IMF, EBF, and BRBF).

Provisions. Refers to this document, the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341).

Quality assurance plan. Written description of qualifications, procedures, quality inspections, resources and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.

Reduced beam section. Reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.

Reinforced-concrete-encased shapes. Structural steel sections encased in reinforced concrete.

*Required strength**. Forces, stresses, and deformations acting on a structural component, determined by either structural analysis, for the *LRFD or ASD* *load combinations*, as appropriate, or as specified by the *Specification* and these *Provisions.*

Restraining bars. Steel reinforcement in composite members that is not designed to carry required loads, 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.

Resistance factor, ϕ†. Factor that accounts for unavoidable deviations of the *nominal strength* from the actual strength and for the manner and consequences of failure.

Safety factor, Ω†. Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the *nominal load*, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Seismic design category. Classification assigned to a building by the *applicable building code* based upon its *occupancy category* and the design spectral response acceleration coefficients.

Seismic force resisting system (SFRS). That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed in SEI/ASCE 7.

Seismic response modification coefficient, *R*. Factor that reduces seismic load effects to strength level as specified by the *applicable building code.*

Special concentrically braced frame (SCBF). Diagonally braced frame meeting the requirements of Section F2 in which all members of the braced-frame system are subjected primarily to axial forces.

Special moment frame (SMF). Moment frame system that meets the requirements of Section E3.

Special plate shear wall (SPSW). Plate shear wall system that meets the requirements of Section F5.

Special seismic systems. Seismic systems designed assuming significant inelastic action occurs in some members under the design earthquake.

Special truss moment frame (STMF). Truss moment frame system that meets the requirements of Section E4.

Specification. Refers to the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360).

Steel core. Axial-force-resisting element of diagonal braces in BRBF. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.

Subassemblage test specimen. The combination of the brace, the connections and testing apparatus that replicate as closely as practical the axial and flexural deformations of the brace in the *prototype*.

Tested connection. Connection that complies with the requirements of Section J2.

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

Test specimen. Brace test specimen or subassemblage test specimen.

Test subassemblage. The combination of the test specimen and pertinent portions of the test setup.

Total link rotation angle. The relative displacement of one end of the link with respect to the other end (measured transverse to the longitudinal axis of the undeformed link), divided by the link length. The total link rotation angle includes both elastic and inelastic components of deformation of the link and the members attached to the link ends.

V-braced frame. Concentrically braced frame (SCBF, OCBF, BRBF, C-OBF or C-SCBF) 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*.

Vertical boundary element (VBE). A column with a connection to one or more web plates in a SPSW.

X-braced frame. Concentrically braced frame (OCBF, SCBF, C-OBF or C-SCBF) in which a pair of diagonal braces crosses near the mid-length of the diagonal braces.

Y-braced frame. *Eccentrically braced frame* (EBF and C-EBF) in which the stem of the Y is the *link* of the EBF system.

ACRONYMS

The following acronyms appear in the AISC *Seismic Provisions for Structural Steel Buildings*. The acronyms are written out where they first appear within a Section.

ACI (American Concrete Institute)
ANSI (American National Standards Institute)
ASCE (American Society of Civil Engineers)
ASD (allowable strength design)
ASNT (American Society for Nondestructive Testing)
ASTM (American Society for Testing of Materials)
AWS (American Welding Society)
BRBF (buckling-restrained braced frame)
CAC-A (air carbon arc cutting)
C-CBF (composite special concentrically braced frame)
CCS (cantilever column system)
C-EBF (composite eccentrically braced frame)
C-IMF (composite intermediate moment frame)
CJP (complete joint penetration)
C-OBF (composite ordinary braced frame)
C-OMF (composite ordinary moment frame)
C-ORCW (composite ordinary reinforced concrete shear wall with steel elements)
C-PRMF (composite partially restrained moment frame
CPRP (connection prequalification review panel)
C-SCBF (composite special concentrically braced frame)
C-SMF (composite special moment frame)
C-SPW (composite steel plate shear wall)
C-SRCW (composite special reinforced concrete shear walls with steel elements)
CVN (Charpy V-notch)
EBF (eccentrically braced frame)
FCAW (flux cored arc welding)
FEMA (Federal Emergency Management Agency)
GMAW (gas metal arc welding)
HBE (horizontal boundary element)
HSS (hollow structural section)
IBE (intermediate boundary element)
IMF (intermediate moment frame)
LAST (lowest anticipated service temperature)
LRFD (load and resistance factor design)
MT (magnetic particle testing)
NDT (nondestructive testing)
OCBF (ordinary concentrically braced frame)
OMF (ordinary moment frame)
QA (quality assurance)
QC (quality control)

PJP (partial joint penetration)
RBS (reduced beam section)
RCSC (Research Council on Structural Connections)
SAW (submerged arc welding)
SCBF (special concentrically braced frame)
SDC (seismic design category)
SFRS (seismic force resisting system)
SMAW (shielded metal arc welding).
SMF (special moment frame)
SPSW (special plate shear wall)
SRC (steel-reinforced concrete)
STMF (special truss moment frame)
UT (ultrasonic testing)
VBE (vertical boundary element)
WPQR (welder performance qualification records)
WPS (welding procedure specifications)

CHAPTER A

GENERAL REQUIREMENTS

This chapter states the scope of the *Provisions*, summarizes referenced specification, code, and standard documents, and provides requirements for materials and contract documents.

The chapter is organized as follows:

A1. Scope

A2. Referenced Specifications, Codes and Standards

A3. Materials

A4. Drawings And Specifications

A1. SCOPE

The *Seismic Provisions for Structural Steel Buildings*, hereafter referred to as these Provisions, shall govern the design, fabrication and erection of structural steel members and connections in the *seismic force resisting systems* (SFRS), and splices and bases of columns in gravity framing systems of buildings and other structures with *moment frames*, *braced frames* and *shear walls*. Other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral force-resisting-elements. These *Provisions* shall apply for design of seismic force resisting systems of structural steel or of structural steel acting compositely with reinforced concrete, unless specifically exempted by the applicable building code.

Wherever these provisions refer to the *applicable building code* and there is no local building code, the loads, *load combinations*, system limitations, and general design requirements shall be those in SEI/ASCE 7.

User Note: SEI/ASCE 7 (Table 12.2-1, Item H) specifically exempts structural steel systems, but not composite systems, in Seismic Design Categories B and C if they are designed according to the *Specification for Structural Steel Buildings* and the seismic loads are computed using an *R* factor of 3. For Seismic Design Category A, SEI/ASCE 7 does specify lateral forces to be used as the seismic loads and effects, but these calculations do not involve the use of an *R* factor. Thus for Seismic Design Category A it is not necessary to define a seismic force resisting system that meets any special requirements and these *Provisions* do not apply.

User Note: Composite seismic force resisting systems include those systems with at least some members of structural steel acting compositely with reinforced concrete, as well as systems in which structural steel members and reinforced concrete members act together to be a composite system.

These *Provisions* shall be applied in conjunction with the AISC *Specification for Structural Steel Buildings*, hereafter referred to as the *Specification*. Members and connections of the SFRS shall satisfy the requirements of the applicable building code, the *Specification*, and these *Provisions*.

Building Code Requirements for Structural Concrete (ACI 318), as modified in these *Provisions*, shall be used for the design of reinforced concrete components in composite construction. For the SFRS in composite construction incorporating reinforced concrete components designed according to ACI 318, the requirements of *Specification* Section B3.3, Design for Strength Using Load and Resistance Factor Design, shall be used.

When the design is based upon elastic analysis, the stiffness properties of the component members of composite construction shall reflect their condition at the onset of significant yielding of the structure.

[SECTION A2 BELOW IS NOT INCLUDED IN THE APRIL 09 BALLOT]

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The documents referenced in these *Provisions* shall include those listed in *Specification* Section A2 with the following additions:

American Concrete Institute (ACI)
Details and Detailing of Concrete Reinforcement, ACI 315-99
Manual of Engineering and Placing Drawings for Reinforced Concrete Structures, ACI 315-R-94
ACI Detailing Manual, ACI SP-66
Monolithic Joints in Concrete Structures, ACI 352

American Institute of Steel Construction (AISC)
Specification for Structural Steel Buildings, ANSI/AISC 360-2010
Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, ANSI/AISC 358-10, including Supplement No. 1

American Welding Society (AWS)

Structural Welding Code–Seismic Supplement, ANSI/AWS D1.8/D1.8M:2009

Standard Methods for Mechanical Testing of Welds- U.S. Customary, ANSI/AWS B4.0-98

Standard Methods for Mechanical Testing of Welds – Metric Only, ANSI/AWS B4.0M:2000

Structural Welding Code-Reinforcing Steel, AWS D1.4-2005

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A3. MATERIALS

1. Material Specifications

Structural steel used in the *seismic force resisting system* (SFRS) shall meet the requirements of *Specification* Section A3.1a, except as modified in these *Provisions*. The specified minimum yield stress of steel to be used for members in which inelastic behavior is expected shall not exceed 50 ksi (345 MPa) for systems defined in Chapters E, F, G and H except that for systems defined in Sections E1, F1, G1, H1 and H4 this limit shall not exceed 55 ksi (380 MPa). Either of these specified minimum yield stress limits are permitted to be exceeded when the suitability of the material is determined by testing or other rational criteria. This limitation does not apply to columns or other members which are not the primary intended yielding element(s) as indicated in the section, Basis of Design, for the given system.

User Note: Specified minimum yield stress of columns in *special moment frame* (SMF) systems may exceed 50 ksi (345 MPa) if inelastic yielding at the column base is anticipated, as the columns are not a primary intended yielding element.

The structural steel used in the SFRS described in Chapters E, F, G and H shall meet one of the following ASTM Specifications:

A36/A36M

A53/A53M

A500 (Gr. B or C)

A501

A529/A529M

A572/A572M [Gr. 42 (290), 50 (345) or 55 (380)]

A588/A588M

A913/A913M [Gr. 50 (345), 60 (415) or 65 (450)]

A992/A992M

A1011 HSLAS Gr. 55 (380)

A1043/A1043M

The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D.

ASTM A572/A572M Gr. 42 (290) is not permitted in members where the load effect of the member or its connections is based upon the *required strength* of the member as determined per Section A3.2.

Other steels and nonsteel materials in *buckling-restrained braced frames* are permitted to be used subject to the requirements of Sections F4 and J3.

User Note: This section only covers material properties for structural steel used in the SFRS and included in the definition of structural steel given in Section 2.1 of the AISC *Code of Standard Practice*. Other steel, such as cables for permanent bracing, is not covered.

2. Expected Material Strength

When required in these *Provisions*, the required strength of an element (a member or a connection of a member) shall be determined from the *expected yield stress*, R_yF_y, of the member or an adjoining member, as applicable, where F_y is the specified minimum yield stress of the grade of steel to be used in the member and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y, of that material.

When required to determine the *nominal strength*, R_n, for limit states within the same member from which the required strength is determined, the expected yield stress, R_yF_y, and the *expected tensile strength*, R_tF_u, are permitted to be used in lieu of F_y and F_u, respectively, where F_u is the specified minimum tensile strength and R_t is the ratio of the expected tensile strength to the specified minimum tensile strength, F_u, of that material.

User Note: In several instances a member, or a connection limit state within that member, is required to be designed for forces corresponding to the expected strength of the member itself. Such cases include determination of the nominal strength, R_n, of the beam outside of the link in EBF, diagonal brace rupture limit states (block-shear rupture and net-section rupture in the diagonal brace in SCBF), etc. In such cases it is permitted to use the expected material strength in the determination of available member strength. For connecting elements and for other members, specified material strength should be used.

The values of R_y and R_t for various steel and concrete materials are given in Table A3.1. Other values of R_y and R_t shall be permitted if the values are determined by testing of specimens, similar in size and source to the materials to be used, conducted in accordance with the testing requirements per the ASTM specifications for the specified grade of steel.

TABLE A3.1

R_y and R_t Values for Steel and Concrete Materials

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M,	1.5	1.2
• ASTM A1043/1043M Gr. 36 (248)	1.3	1.1
• ASTM A572/572M Gr. 50 (345) or 55 (380), ASTM A913/A913M Gr. 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M,	1.1	1.1
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
• ASTM A529 Gr. 50 (345)	1.2	1.2
• ASTM A529 Gr. 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500 (Gr. B or C), • ASTM A501	1.4	1.3
Pipe:		
• ASTM A53/A53M	1.6	1.2
Plates, Strips and Sheets:		
• ASTM A36/A36M,	1.3	1.2
• ASTM A1043/1043M Gr. 36 (248)	1.3	1.1
• A1011 HSLAS Gr. 55 (380),	1.1	1.1
• ASTM A572/A572M Gr. 50 (345), Gr. 55 (380), ASTM A588/A588M,	1.1	1.2
• ASTM 1043/1043M Gr. 50 (345)	1.2	1.1
• Steel Reinforcement: • ASTM A615, • ASTM A706 •	1.25	1.25
• Structural Concrete: • Structural Concrete	See ACI	See ACI

•	318 Table 5.3.2.2	318 Table 5.3.2.2

3. Heavy Sections

For structural steel in the SFRS, in addition to the requirements of *Specification* Section A3.1c, hot rolled shapes with flanges 1½ in. thick (38 mm) and thicker shall have a minimum Charpy V-notch toughness of 20 ft-lbs (27 J) at 70 °F (21 °C), tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates 2 in. (50 mm) thick and thicker shall have a minimum Charpy V-notch toughness of 20 ft-lbs (27 J) at 70 °F (21 °C), measured at any location permitted by ASTM A673, Frequency P, where the plate is used in the following:

(a) Members built-up from plate

(b) Connection plates where inelastic strain under seismic loading is expected

(c) The *steel core* of buckling restrained braces

User Note: Examples of connection plates where inelastic behavior is expected include, but are not limited to, gusset plates intended to function as a hinge and allow out-of-plane buckling of diagonal braces, some bolted flange plates for moment connections, some end plates for bolted moment connections, and some column base plates designed as a pin.

4. Consumables for Welding

4a. Seismic Force Resisting System Welds

All welds used in members and connections in the SFRS shall be made with filler metals meeting the requirements specified in AWS D1.8 Clause 6.3. AWS D1.8 Clauses 6.3.5, 6.3.6, 6.3.7 and 6.3.8 shall apply only to demand critical welds..

4b. Demand Critical Welds

Welds designated as demand critical shall be made with filler metals meeting the requirements specified in AWS D1.8 Clause 6.3.

User Note:
AWS D1.8 requires that all seismic force resisting system welds are to be made with filler metals classified using AWS A5 standards that achieve the following mechanical properties:

Filler Metal Classification Properties for Seismic Force Resisting System Welds

Property	Classification: 70 ksi (480 MPa)	Classification: 80 ksi (550 MPa)
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.
Elongation, %	22 min.	19 min.
CVN Toughness, ft·lbf (J)	20 (27) min. @ 0 °F (–18 °C) [a]	

[a] Filler metals classified as meeting 20 ft-lbf (27 J) min. at a temperature lower than 0 °F (–18 °C) also meet this requirement.

In addition to the above requirements, AWS D1.8 requires, unless otherwise exempted from testing, that all demand critical welds are to be made with filler metals receiving Heat Input Envelope Testing that achieve the following mechanical properties in the weld metal:

Mechanical Properties for Demand Critical Welds

Property	Classification: 70 ksi (480 MPa)	Classification: 80 ksi (550 MPa)
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.
Elongation (%)	22 min.	19 min.
CVN Toughness, ft·lbf (J)	40 (54) min. @ 70 °F (20 °C) [b, c]	

[b] For LAST of +50 °F (+10 °C). For LAST less than + 50 °F (+10 °C), see AWS D1.8 Clause 6.3.6.
[c] Tests conducted in accordance to AWS D1.8 Annex A meeting 40 ft-lbf (54 J) min. at a temperature lower than +70 °F (20 °C) also meet this requirement.

5. Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite components in composite intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5 and H6 shall meet the requirements of ACI 318, Chapter 21. Concrete and steel reinforcement used in composite components in composite ordinary SFRS of Sections G1, H1 and H4 shall meet the requirements of ACI 318, Section 21.1.1.5.

A4. DRAWINGS AND SPECIFICATIONS

1. General

Structural design drawings and specifications shall indicate the work to be performed, and include items required by the *Specification*, the AISC *Code of Standard Practice for Steel Buildings and Bridges*, the *applicable building code*, and the following, as applicable:

(1) Designation of the SFRS.

(2) Identification of the members and connections that are part of the SFRS.

(3) Locations and dimensions of *protected zones.*

(4) Connection details between the diaphragm and the SFRS, when concrete floor slabs serve as diaphragms.

(5) Shop drawing and erection drawing requirements not addressed in Section I1.

User Note: For connections and applications for which details are not specifically addressed by the *Provisions*, The *engineer of record* should include applicable requirements in the design drawings and specifications. Any requirements for shop drawings and erection drawings beyond the requirements of these *Provisions* should be clearly indicated in the design drawings and specifications.

2. Steel Construction

In addition to the requirements of Section A4.1, structural design drawings and specifications for steel construction shall indicate the following items, as applicable:

(1) Configuration of the connections.

(2) Connection material specifications and sizes.

(3) Locations of *demand critical welds.*

(4) Locations where gusset plates are to be detailed to accommodate inelastic rotation.

(5) Locations of connection plates requiring Charpy V-notch (CVN) toughness in accordance with Section A3.3(b).

(6) *Lowest anticipated service temperature* (LAST) of the steel structure, if the structure is not enclosed and maintained at a temperature of 50 °F (10 °C) or higher.

(7) Locations where weld backing is required to be removed.

(8) Locations where fillet welds are required when weld backing is permitted to remain.

(9) Locations where fillet welds are required to reinforce groove welds or to improve connection geometry.

(10) Locations where weld tabs are required to be removed.

(11) Splice locations where tapered transitions are required.

(12) The shape of weld access holes, if a shape other than those provided for in the *Specification* is required.

(13) Joints or groups of joints in which a specific assembly order, welding sequence, welding technique or other special precautions are required to be submitted to the engineer of record.

3. Composite Construction

In addition to the requirements of Section A4.1, and the requirements of Section A4.2 as applicable for the steel components of reinforced concrete or composite elements, structural design drawings, and specifications for composite construction shall indicate the following items, as applicable:

(1) Bar placement, cutoffs, lap and mechanical splices, hooks and mechanical anchorage, placement of ties and other transverse reinforcement.

(2) Requirements for dimensional changes resulting from temperature changes, creep and shrinkage.

(3) Location, magnitude and sequencing of any prestressing or post-tensioning present.

(4) Location of steel headed stud anchors and welded reinforcing bar anchors.

CHAPTER B

GENERAL DESIGN REQUIREMENTS

This chapter addresses the general requirements for the seismic design of steel structures that are applicable to all chapters of the *Provisions*.

This chapter is organized as follows:

B1. GENERAL SEISMIC DESIGN REQUIREMENTS

The *required strength* and other seismic provisions for *seismic design categories* (SDCs), *occupancy categories*, and the limitations on height and irregularity shall be as specified in the *applicable building code*.

The *design story drift* and the limitations on story drift shall be determined as required in the *applicable building code*.

B2. LOADS AND LOAD COMBINATIONS

The loads and *load combinations* shall be as stipulated by the *applicable building code*. Where *amplified seismic loads* are required by these *Provisions*, the seismic load effect including the overstrength factor, E_m, shall be applied as prescribed by the applicable building code

For the *seismic force resisting system* (SFRS), in composite construction, incorporating reinforced concrete components designed according to the requirements of ACI 318, the requirements of *Specification* Section B3.3, Design for Strength Using Load and Resistance Factor Design, shall be used.

User Note: When not defined in the applicable building code, Ω_o should be taken from SEI/ASCE 7.

B3. DESIGN BASIS

1. Required Strength

The *required strength* of structural members and connections shall be the greater of:

(1) The required strength as determined by structural analysis for the appropriate *load combinations*, as stipulated in the *applicable building code*, and in Chapter C.

(2) The required strength given in Chapters D, E, F, G and H.

2. Available Strength

The *available strength* is stipulated as *design strength*, ϕR_n, for design according to the provisions for *load and resistance factor design* (LRFD) and *allowable strength*, R_n / Ω, for design according to the provisions for *allowable strength design* (ASD). The available strength of systems, members, and connections shall be determined according to the *Specification*, except as modified throughout these *Provisions*.

B4. SYSTEM TYPE

The seismic force resisting system (SFRS) shall contain one or more moment frame, braced frame or shear wall systems conforming to the requirements of one of the seismic systems designated in Chapters E, F, G and H.

CHAPTER C

ANALYSIS

This chapter addresses design related analysis requirements. The chapter is organized as follows:

C1. General

C2. Additional Requirements

C3. Nonlinear Analysis

C1. GENERAL

An analysis conforming to the requirements of the *applicable building code* and the *Specification* shall be performed for design of the system.

When the design is based upon elastic analysis, the stiffness properties of component members of steel systems shall be based on elastic sections and those of composite systems shall include the effects of cracked sections.

C2. ADDITIONAL REQUIREMENTS

Additional analysis shall be performed as specified in Chapters E, F, G and H of these *Provisions*.

C3. NONLINEAR ANALYSIS

When nonlinear analysis is used to satisfy the requirements of these *Provisions*, it shall be performed in accordance with Chapter 16 of SEI/ASCE 7.

CHAPTER D

GENERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS

This chapter addresses general requirements for the design of members and connections.

The chapter is organized as follows:

D1. Member Requirements
D2. Connections
D3. Deformation Compatibility of Non-SRFS Members and Connections
D4. H-Piles

D1. MEMBER REQUIREMENTS

Members of moment frames, braced frames and shear walls in the seismic force resisting system (SFRS) shall comply with the *Specification* and this section.

1. Classification of Sections for Ductility

Certain members of the SFRS that are expected to undergo inelastic deformation under the design earthquake are designated in these provisions as *moderately ductile members* or *highly ductile members*. When required for the systems defined in Chapters E, F, G, H and Section D4, these members shall comply with this section.

1a. Section Requirements for Ductile Members

Structural steel sections for both *moderately ductile members* and *highly ductile members* shall have flanges continuously connected to the web or webs. Webs and flanges of built-up sections shall be continuously connected by fillet or groove welds. Welds of webs to flanges of built-up sections used as link beams in *eccentrically braced frames* shall be complete-joint-penetration groove welds. Built-up sections used in *intermediate moment frames* and *special moment frames* shall comply with Section 2.3.2 of ANSI/AISC 358.

Encased structural steel sections shall comply with the requirements of Section D1.4b(1) for moderately ductile members and Section D1.4b(2) for highly ductile members.

Filled composite sections shall comply with the requirements of Section D1.4c for both moderately and highly ductile members.

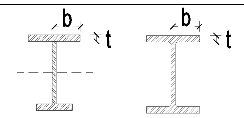
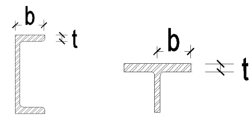
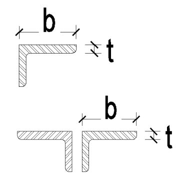
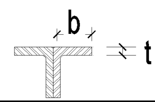
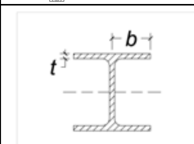
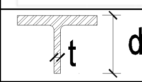
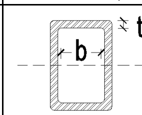
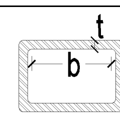
Concrete sections shall comply with the requirements of ACI 318 Section 21.3 for moderately ductile members and ACI 318 Section 21.6 for highly ductile members.

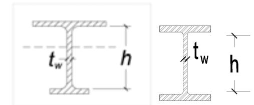
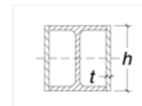
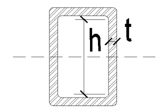
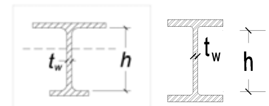
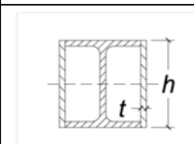
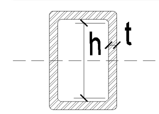
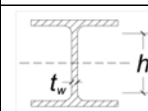
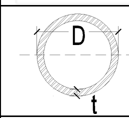
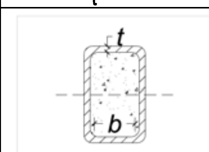
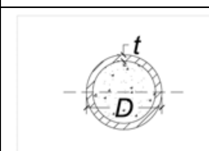
1b. Width-to-Thickness Limitations of Steel Sections

For members designated as *moderately ductile members*, the width-thickness ratios of compression elements shall not exceed the limiting width-thickness ratios, λ_{md}, from Table D1.1.

For members designated as *highly ductile members*, the width-thickness ratios of compression elements shall not exceed the limiting width-thickness ratios, λ_{hd}, from Table D1.1.

TABLE D1.1
Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratio λ_{hd} Highly Ductile Members	 λ_{md} Moderately Ductile Members	Example
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees				
	Legs of single angles or double angle members with separators	*b/t*	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$	
	Outstanding legs of pairs of angles in continuous contact				
	Flanges of H-pile sections per Section D4	*b/t*	$0.45\sqrt{E/F_y}$	not applicable	
	Stems of tees [a]	*d/t*	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$	
Stiffened Elements	Flanges of boxed I-shaped sections and built-up box sections	*b/t*	$0.60\sqrt{E/F_y}$	$1.12\sqrt{E/F_y}$	
	Walls of rectangular HSS	*b/t*	$0.55\sqrt{E/F_y}$	$0.64\sqrt{E/F_y}$ [b]	

	Members subject to flexure or combined flexure and compression:				
	Webs of rolled or built-up I-shaped sections,[d]	h/t_w	For $C_a \le 0.125$ $2.45\sqrt{E/F_y}\left(1-0.93C_a\right)$ For $C_a > 0.125$ $0.77\sqrt{E/F_y}\ (2.93-C_a)$ $\ge 1.49\sqrt{E/F_y}$ where $C_a = \dfrac{P_u}{\phi_b P_y}$ (LRFD) $C_a = \dfrac{\Omega_b P_a}{P_y}$ (ASD)	For $C_a \le 0.125$ $3.76\sqrt{E/F_y}\ (1-2.75C_a)$ For $C_a > 0.125$ $1.12\sqrt{E/F_y}\ (2.33-C_a)$ $\ge 1.49\sqrt{E/F_y}$ where $C_a = \dfrac{P_u}{\phi_b P_y}$ (LRFD) $C_a = \dfrac{\Omega_b P_a}{P_y}$ (ASD)	
	Side plates of boxed I-shaped sections,	h/t			
	Webs of built-up box sections	h/t			
	Webs of rolled or built-up I-shaped sections subject to uniform compression	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$	
	Side plates of boxed I-shaped sections, and webs of built-up box sections subject to uniform compression[e]	h/t	Columns: $1.49\sqrt{E/F_y}$ Diagonal Braces: $0.60\sqrt{E/F_y}$	Columns: $1.49\sqrt{E/F_y}$ Diagonal Braces: $0.70\sqrt{E/F_y}$	 
	Webs of H-Pile sections	h/t_w	$0.94\sqrt{E/F_y}$	not applicable	
	Walls of round HSS	D/t	$0.038E/F_y$	$0.044E/F_y$ [c]	
Composite Elements	Rectangular filled composite member	b/t	$1.4\sqrt{E/F_y}$	$2.26\sqrt{E/F_y}$	
	Round filled composite member	D/t	$0.076E/F_y$	$0.15\ E/F_y$	

[a] For tee shaped compression members, the limiting width-to-thickness ratio for the stem of the tee can be increased to $0.38\sqrt{E/F_y}$ if either of the following conditions are satisfied:

(1) Buckling of the compression member occurs about the plane of the stem.
(2) The compression load is transferred at end connections to only the outside face of the flange of the tee resulting in reduced compression stresses at the tip of the stem of the tee due to flexure from the eccentric connection.

[b] The limiting width to thickness ratio of walls of rectangular *HSS* members used as beams or columns shall not exceed $1.12\sqrt{E/F_y}$

[c] The limiting diameter to thickness ratio of round HSS members used as beams or columns shall not exceed $0.07\ E/F_y$.

[d] For beams in OMF, IMF or SMF systems, where C_a is less than 0.125, the limiting ratio h/t_w shall not exceed $2.45\sqrt{E/F_y}$.

2. Stability Bracing of Members

Where required in Chapters E, F, G and H, stability bracing of structural steel, concrete-encased or concrete-filled members subject to flexure shall be provided to restrain lateral-torsional buckling as required in this section.

User Note: In addition to the requirements in Chapters E, F, G and H to provide stability bracing for various beam members such as intermediate and special moment frame beams, stability bracing is also required for columns in the *cantilever column systems* (CCS) in Sections E5 and E6.

2a. Moderately Ductile Members

(1) The bracing of moderately ductile steel members shall satisfy the following requirements:

(i) Both flanges of members shall be laterally braced or the member cross section shall be torsionally braced.

(ii) Member bracing shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams, where the expected flexural strength of the member shall be:

$$M_r = M_u = R_y Z F_y \text{ (LRFD)} \quad \text{(D1-1a)}$$

or

$$M_r = M_a = R_y Z F_y / 1.5 \text{ (ASD)} \quad \text{(D1-1b)}$$

where

$C_d = 1.0$

(iii) Member bracing shall have a maximum spacing of

$$L_b = 0.17 r_y E / F_y \quad \text{(D1-2)}$$

(2) The bracing of moderately ductile concrete-encased and concrete-filled composite members shall satisfy the following requirements:

(i) Both flanges of members shall be laterally braced or the member cross-section shall be torsionally braced..

(ii) Lateral bracing shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams, where $M_r = M_u = M_p$ of the member as specified in Section G2.6c, and $C_d = 1.0$.

(iii) Member bracing shall have a maximum spacing of

$$L_b = 0.17 r_y E/F_y \tag{D1-3}$$

using the properties of the steel section, except that in the calculation for concrete encased composite members, r_y shall not be taken as less than 0.3 times the overall depth of the cross section in the plane of buckling.

2b. Highly Ductile Members

In addition to the requirements of Sections D1.2a(1)(i) and (ii), and D1.2a(2)(i) and (ii), the bracing of *highly ductile members* shall have a maximum spacing of $L_b = 0.086 r_y E/F_y$. For concrete-encased and concrete-filled composite members, the properties of the steel section shall be used, except that in the calculation for concrete encased composite members, r_y shall not be taken as less than 0.3 times the overall depth of the cross section in the plane of buckling.

2c. Special Bracing at Plastic Hinge Locations

Special bracing shall be located adjacent to expected plastic hinge locations where required by Chapters E, F, G or H.

(1) For structural steel members, such bracing shall satisfy the following requirements:

(i) Both flanges of members shall be laterally braced or the member cross section shall be torsionally braced..

(ii) The *required strength* of lateral bracing of each flange provided adjacent to plastic hinges shall be:

$$P_u = 0.06\, R_y Z F_y / h_o \text{ (LRFD)} \tag{D1-4a}$$

or

$$P_a = (0.06/1.5)\, R_y Z F_y / h_o \text{ (ASD)} \tag{D1-4b}$$

where

h_o = distance between flange centroids, in.

The *required strength* of torsional bracing provided adjacent to plastic hinges shall be:

$$M_u = 0.06\, R_y Z F_y \text{ (LRFD)} \tag{D1-5a}$$

or

$$M_a = 0.06\, R_y\, Z\, F_y \text{ (ASD)} \qquad \text{(D1-5b)}$$

(iii) The required bracing stiffness shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams with C_d =1.0 and where the expected flexural strength of the member shall be:

$$M_r = M_u = R_y\, Z\, F_y \text{ (LRFD)} \qquad \text{(D1-6a)}$$

or

$$M_r = M_a = R_y Z F_y / 1.5 \text{ (ASD)} \qquad \text{(D1-6b)}$$

(2) For concrete-encased and concrete-filled composite members, such bracing shall satisfy the following requirements:

(i) Both flanges of members shall be laterally braced or the member cross section shall be torsionally braced.

(ii) The *required strength* of lateral bracing provided adjacent to plastic hinges shall be

$$P_u = 0.06 M_p / h_o \qquad \text{(D1-7)}$$

of the member, where M_p is determined in accordance with Section G2.6c.

The required strength for torsional bracing provided adjacent to plastic hinges shall be M_u = 0.06 M_p of the member.

(iii) The required bracing stiffness shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams where $M_r = M_u = M_p$ of the member is determined in accordance with Section G2.6c, and C_d = 1.0.

3. Protected Zones

Discontinuities from fabrication and erection procedures and from other attachments are prohibited in the area of a member or a connection element designated as a *protected zone* by these *Provisions* or ANSI/AISC 358. See Section I2.1 for specific limitations.

Exception: Welded shear studs and other connections are permitted in protected zones when designated in ANSI/AISC 358, or as otherwise determined with a connection prequalification in accordance with Section J1, or as determined in a program of qualification testing in accordance with Sections J2 and J3.

4. Columns

Columns in moment frames, braced frames and shear walls shall satisfy the requirements of this section.

4a. Required Strength

The *required strength* of columns in the SFRS shall satisfy both of the following:

(1) The load effect resulting from the analysis requirements for the applicable system per Sections E, F, G and H.

(2) The compressive axial strength and tensile strength as determined using the load combinations stipulated in the applicable building code including the amplified seismic load. It is permitted to neglect applied moments in this determination unless the moment results from a load applied to the column between points of lateral support. The required axial compressive strength and tensile strength need not exceed either of the following:

(a) The maximum load transferred to the column by the system, including the effects of material overstrength and strain hardening.

(b) The forces corresponding to the resistance of the foundation to overturning uplift.

4b. Encased Composite Columns

Encased composite columns shall satisfy the requirements of *Specification* Chapter I, in addition to the requirements of this section. Additional requirements, as specified for *moderately ductile members* and *highly ductile members* in Sections D1.4b(1) and (2) shall apply as required in the descriptions of the composite seismic systems in Chapters G and H.

(1) Moderately Ductile Members

Encased composite columns used as moderately ductile members shall satisfy the following requirements:

(i) The maximum spacing of transverse reinforcement 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. (300 mm)

(ii) This spacing 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. (450 mm)

(iii) Tie spacing over the remaining column length shall not exceed twice the spacing defined in Section D1.4b(1)(i)..

(iv) Splices and end bearing details for *encased composite columns* in ordinary seismic systems shall meet the requirements of the *Specification* and ACI 318 Section 7.8.2. The design shall comply with ACI 318 Sections 21.1.6 and 21.1.7. The design shall consider any adverse behavioral effects due to abrupt changes in either the member stiffness or the nominal *tensile strength*. Locations of such adverse behavioral effects shall include transitions to reinforced concrete sections without embedded structural steel members, transitions to bare structural steel sections, and column bases.

(v) Welded wire fabric shall be prohibited as transverse reinforcement in *moderately ductile members*.

(2) Highly Ductile Members

Encased composite columns used as highly ductile members shall satisfy Section D1.4b(1) in addition to the following requirements:

(i) Longitudinal *load-carrying reinforcement* shall meet the requirements of ACI 318 Section 21.6.3.

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

(1) The minimum area of tie reinforcement, A_{sh}, shall be:

$$A_{sh} = 0.09 h_{cc} s \left(1 - \frac{F_y A_s}{P_n}\right)\left(\frac{f'_c}{F_{yh}}\right) \quad \text{(D1-8)}$$

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 stress of the structural *steel core*, ksi (MPa)

A_s = cross-sectional area of the structural steel core, in.^{2} (mm^2)

P_n = nominal compressive strength of the composite column calculated in accordance with the *Specification*, kips (N)

f'_c = specified compressive strength of concrete, ksi (MPa)

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

Equation D1-8 need not be satisfied if the *nominal strength* of the concrete-encased structural steel section alone is greater than the *load effect* from a load combination of 1.0D+0.5L.

(2) The maximum spacing of transverse reinforcement along the length of the column shall be the lesser of six longitudinal load-carrying bar diameters or 6 in. (150 mm).

(3) When specified in Sections D1.4b(1)(ii), (iii) or (iv), the maximum spacing of transverse reinforcement along the member length shall be the lesser of one-fourth the least member dimension or 4 in. (100 mm). Confining reinforcement shall be spaced not more than 14 in. (350 mm) on center in the transverse direction.

(iii) *Encased composite columns* in braced frames with required compressive strengths , without consideration of the amplified seismic loads, greater than 0.2P_n shall have transverse reinforcement as specified in Section D1.4b(2)(ii)(3) over the total element length. This requirement need not be satisfied if the *nominal strength* of the concrete-encased steel section alone is greater than the *load effect* from a load combination of 1.0D+0.5L.

(iv) *Composite columns* supporting reactions from discontinued stiff members, such as walls or braced frames, shall have transverse reinforcement as specified in Section D1.4b(2)(ii)(3) over the full length beneath the level at which the discontinuity occurs if the required compressive strengths, without consideration of the amplified seismic loads, exceeds 0.1P_n. Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the concrete-encased steel section and longitudinal reinforcement. This requirement need not be satisfied if the *nominal strength* of the concrete-encased steel section alone is greater than the *load effect* from a load combination of 1.0D+0.5L.

(v) *Encased composite columns* used in a C-SMF shall satisfy the following requirements:

(1) Transverse reinforcement shall meet the requirements in Section D1.4b(2)(ii) at the top and bottom of the column over the region specified in Section D1.4b(1).

(2) The strong-column/weak-beam design requirements in Section G3.4a shall be satisfied. Column bases shall be detailed to sustain inelastic flexural hinging.

(3) The *required shear strength* of the column shall meet the requirements of ACI 318 Section 21.6.5.1.

(vi) 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. (300 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 concrete-encased shape and longitudinal reinforcement.

4c. Filled Composite Columns

This section applies to columns that meet the limitations of *Specification* Section I2.2. Such columns shall be designed to meet the requirements of *Specification* Chapter I, except that the *nominal shear strength* of the composite column shall be the *nominal shear strength* of the structural steel section alone, based on its effective shear area.

5. Composite Slab Diaphragms

The design of composite floor and roof slab diaphragms for seismic effects shall meet the following requirements.

5a. Load Transfer

Details shall be provided to transfer loads between the diaphragm and boundary members, collector elements, and elements of the horizontal framing system.

5b. Nominal Shear Strength

The nominal in-plane shear strength of composite diaphragms and concrete slab on 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 nominal shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

D2. CONNECTIONS

1. General

Connections, joints and fasteners that are part of the SFRS shall comply with *Specification* Chapter J, and with the additional requirements of this section.

Splices and bases of columns that are not part of the SFRS shall meet the requirements of Sections D2.5a, D2.5c and D2.6.

Where *protected zones* are designated in connection elements by these *Provisions* or ANSI/AISC 358, they shall meet the requirements of Sections D1.3 and I2.1.

2. Bolted Joints

Bolted joints shall satisfy the following requirements:

(i) All bolts shall be installed as pretensioned high strength bolts. Faying surfaces shall meet the requirements for slip-critical faying surfaces in accordance with *Specification* Section J3.8 with a Class A or better surface.

Exceptions: Connection surfaces are permitted to have coatings with a slip coefficient less than that of a Class A faying surface for the following:

(1) End plate moment connections conforming to the requirements of Section E1, or ANSI/AISC 358

(2) Bolted joints where the load effects due to seismic are transferred either by tension in bolts or by compression bearing but not by shear in bolts.

User Note: The designation of connections as slip critical will meet the requirements for pretensioned bolts and slip-critical faying surfaces but will result in additional requirements for inspection.

(ii) Bolt holes shall be standard holes or short-slotted holes perpendicular to the applied load.

Exception: For diagonal braces specified in Sections F1, F2, F3 and F4, oversized holes are permitted in one connection ply only when the connection is designed as a slip-critical joint for the required brace connection strength in Sections F1, F2, F3 and F4, and where slip is considered a strength limit state per *Specification* Section J3.8.

User Note: Diagonal brace connections with oversized holes must also satisfy other limit states including bolt bearing and bolt shear for the required strength of the connection as defined in Sections F1, F2, F3 and F4. Alternative hole types are permitted if designated in the ANSI/AISC 358, or if otherwise determined in a connection prequalification in accordance with Section J1, or if determined in a program of qualification testing in accordance with Section J2 or Section J3.

(iii) The available shear strength of bolted joints using standard holes shall be calculated as that for bearing-type joints in accordance with *Specification* Sections J3.6 and J3.10. The nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

(iv) Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

User Note: A member force, such as a diagonal brace axial force, must be resisted at the connection entirely by one type of joint (in other words, either entirely by bolts or entirely by welds). A connection in which bolts resist a force that is normal to the force resisted by welds, such as a moment connection in which welded flanges transmit flexure and a bolted web transmits shear, is not considered to be sharing the force.

3. Welded Joints

Welded joints shall be designed in accordance with Chapter J of the *Specification*. Welding shall be performed in accordance with Section I2.3. Welding materials shall be in accordance with Section A3.4.

4. Continuity Plates and Stiffeners

The design of *continuity plates* and stiffeners located in the webs of rolled shapes shall allow for the reduced contact lengths to the member flanges and web based on the corner clip sizes in Section I2.4.

5. Column Splices

5a. Location of Splices

For all building columns, including those not designated as part of the SFRS, column splices shall be located 4 ft (1.2 m) or more away from the beam-to-column flange connections. When the column clear height between beam-to-column flange connections is less than 8 ft (2.4 m), splices shall be at half the clear height.

Exceptions:

1) Column splices with webs and flanges joined by complete-joint-penetration groove welds are permitted to be located closer to the beam-to-column flange connections, but not less than the depth of the column.

2) Splices in composite columns.

User Note: Where possible, splices should be located at least 4 ft (1.2m) above the finished floor elevation to permit installation of perimeter safety cables prior to erection of the next tier and to improve accessibility.

5b. Required Strength

The *required strength* of column splices in the SFRS shall be the greater of:

(a) The required strength of the columns, including that determined from Chapters E, F, G and H, and Section D1.4a; or,

(b) The required strength determined using the *load combinations* stipulated in the *applicable building code* including the *amplified seismic load.* The required strength need not exceed the maximum loads that can be transferred to the splice by the system.

In addition, welded column splices in which any portion of the column is subject to a calculated net tensile load effect determined using the load combinations stipulated in the applicable building code, including the amplified seismic load, shall satisfy all of the following requirements:

(1) The *available strength* of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200% of the required strength.

(2) The available strength for each flange splice shall be at least equal to $0.5R_yF_yb_ft_f$ (LRFD) or $(0.5/1.5)R_yF_yb_ft_f$ (ASD), as applicable, where R_yF_y is the *expected yield stress* of the column material and b_ft_f is the area of one flange of the smaller column connected.

(3) Where welded butt joint splices are made with complete-joint-penetration groove welds, tapered transitions are required between flanges of unequal thickness or width per AWS D1.8 Clause 4.2 when tension stress at any location in the smaller flange exceeds $0.30F_y$ (LRFD) or $0.20F_y$ (ASD).

User Note: Butt splices subject to tension greater than 33% of the expected yield strength under any load combination should have tapered transitions. The stress concentration at a nontapered transition, with a 90° corner, could cause local yielding when the tensile stress exceeds 33% of yield. Lower levels of stress would be acceptable with the stress concentration from a nontapered transition.

5c. Required Shear Strength

For all building columns including those not designated as part of the SFRS, the required shear strength of column splices with respect to both orthogonal axes of the column shall be M_{pc}/H (LRFD) or $M_{pc}/1.5H$ (ASD), as applicable, where M_{pc} is the lesser nominal plastic flexural strength of the column sections for the direction in question, and H is the story height.

The required shear strength of splices of columns in the SFRS shall be the greater of the above requirement or the required shear strength determined per Section D2.5b(a) and (b).

5d. Structural Steel Splice Configurations

Structural steel column splices are permitted to be either bolted or welded, or welded to one column and bolted to the other. Splice configurations shall meet any specific requirements in Chapters E, F, G or H.

Where columns of the SFRS use bolted web splices, plates or channels shall be used on both sides of the column web.

For welded butt joint splices made with groove welds, weld tabs shall be removed in accordance with AWS D1.8 Clause 6.10. Steel backing of groove welds need not be removed.

5e. Splices in Encased Composite Columns

For encased composite columns, column splices shall conform to Section D1.4b and ACI 318 Section 21.6.3.2.

6. Column Bases

The *required strength* of column bases, including those that are not part of the SFRS, shall be calculated in accordance with this section

User Note: The only requirement for bases of columns that are not part of the SFRS is the required shear strength per Section D2.5c.

The available strength of steel elements at the column base, including base plates, anchor rods, stiffening plates, and shear lug elements shall be in accordance with the *Specification*.

Where columns are welded to base plates with groove welds, weld tabs and weld backing shall be removed.

The available strength of concrete elements at the column base, including anchor rod embedment and reinforcing steel, shall be in accordance with ACI 318, Appendix D.

User Note: When using concrete reinforcing steel as part of the anchorage embedment design, it is important to understand the anchor failure modes and provide reinforcement that is developed on both sides of the expected failure surface. See ACI 318, Appendix D, including Commentary.

6a. Required Axial Strength

The *required axial strength* of column bases, including their attachment to the foundation, shall be the summation of the vertical components of the required connection *strength*s of the steel elements that are connected to the column base, but not less than the greater of:

(a) The column axial load calculated using the *load combinations* of the *applicable building code*, including the *amplified seismic load.*

(b) The required axial strength for column splices, as prescribed in Section D2.5.

User Note: The vertical components can include both the axial load from columns and the vertical component of the axial load from diagonal members framing into the column base. Section D2.5 includes references to Section D1.4a and Chapters E, F, G and H. Where diagonal braces frame to both sides of a column, the effects of compression brace buckling should be considered in the summation of vertical components. See Section F2.3.

6b. Required Shear Strength

The *required shear strength* of column bases, including their attachments to the foundations, shall be the summation of the horizontal component of the required strengths of the steel elements that are connected to the column base as follows:

(1) For diagonal braces, the horizontal component shall be determined from the required strength of diagonal brace connections for the SFRS.

(2) For columns, the horizontal component shall be equal to the required shear strength for column splices prescribed in Section D2.5c.

Exception: Single story columns with simple end connections at both ends need not comply with D2.6b(2).

User Note: The horizontal components can include the shear load from columns and the horizontal component of the axial load from diagonal members framing into the column base. Section D2.5 includes references to Section D1.4a and Chapters E, F, G and H.

6c. Required Flexural Strength

The *required flexural strength* of column bases, including their attachment to the foundation, shall be the summation of the required connection strengths of the steel elements that are connected to the column base as follows:

(1) For diagonal braces, the *required flexural strength* shall be at least equal to the required flexural strength of diagonal brace connections.

(2) For columns, the *required flexural strength* shall be at least equal to the lesser of the following:

(a) $1.1R_yF_yZ$ (LRFD) or $(1.1/1.5)R_yF_yZ$ (ASD), as applicable, of the column, or

(b) the moment calculated using the *load combinations* of the *applicable building code*, including the *amplified seismic load*.

User Note: Moments at column to column base connections designed as simple connections may be ignored provided the connection has sufficient ductility to accommodate the rotation resulting from the lateral drift of the structure as calculated per the applicable building code.

7. Composite Connections

This section applies to connections in buildings that utilize composite or dual steel and concrete systems wherein seismic load is transferred between structural steel and reinforced concrete components. Methods for calculating the connection strength shall meet the requirements in this section. Unless the connection strength is determined by analysis or testing, the models used for design of connections shall satisfy the following requirements:

(1) Force shall be transferred between structural steel and reinforced concrete through:

(a) direct bearing from internal bearing mechanisms;

(b) shear connection;

(c) shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer; or

(d) a combination of these means.

The contribution of different mechanisms is permitted to be combined only if the stiffness and deformation capacity of the mechanisms are compatible. 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 of ACI 318 Chapters 10 and 11. Unless a higher strength is substantiated by cyclic testing, the nominal bearing and shear-friction strengths shall be reduced by 25% for the composite seismic systems described in Sections G3, H2, H3, H5 and H6.

(3) *Face bearing plates* consisting of stiffeners between the flanges of steel beams shall be provided when beams are embedded in reinforced concrete columns or walls.

(4) The *nominal shear strength* of 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 Section E3.6e and ACI 318 Section 21.7, respectively.

(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 applicable, 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 G3, H2, H3, H5 and H6 shall meet the requirements of ACI 318 Section 21.7.5.

(6) Composite connections shall satisfy the following additional requirements:

(i) 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, diagonal braces and walls.

(ii) For connections between structural steel or *composite beams* and reinforced concrete or *encased composite columns*, transverse hoop reinforcement shall be provided in the connection region of the column to meet the requirements of ACI 318 Section 21.7, except for the following modifications:

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

(2) 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 G1, G2, H1 and H4.

(3) The longitudinal bar sizes and layout in reinforced concrete and *composite columns* are 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. Steel Anchors

Where steel headed stud anchors or welded reinforcing bar anchors are part of the intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5 and H6, their shear and tensile strength shall be reduced by 25% from the specified strengths given in *Specification* Chapter I.

> **User Note:** The 25% reduction is not necessary for gravity and collector components in structures with intermediate or special seismic force resisting systems designed for the amplified seismic load.

D3. DEFORMATION COMPATIBILITY OF NON-SFRS MEMBERS AND CONNECTIONS

Where deformation compatibility of members and connections that are not part of the *seismic force resisting system* (SFRS) is required by the *applicable building code*, these elements shall be designed to resist the combination of gravity load effects and the effects of deformations occurring at the story drift calculated in accordance with the applicable building code.

> **User Note:** SEI/ASCE 7 stipulates the above requirement for structural steel members and connections. Flexible shear connections that allow member end rotations per Section J1.2 of the *Specification* shall be considered to meet these requirements. Inelastic deformations are permitted in connections or member provided they are self limiting and do not create instability in the member. See Commentary for further discussion.

D4. H PILES

1. Design Requirements

Design of H-piles shall comply with the requirements of the *Specification* regarding design of members subjected to combined loads. H-piles shall meet the requirements for *highly ductile members* of Section D1.1

2. Battered H-Piles

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

3. Tension

Tension in each pile shall be transferred to the pile cap by mechanical means such as shear keys, reinforcing bars or studs welded to the embedded portion of the pile.

4. Protected Zone

At each pile, the length equal to the depth of the pile cross section located directly below the bottom of the pile cap shall be designated as a *protected zone* meeting the requirements of Sections D1.3 and I2.1.

CHAPTER E

MOMENT-FRAME SYSTEMS

This chapter provides the basis of design and the requirements pertaining to system configuration, members and connections for steel ordinary, intermediate, special and special truss moment frames, and for steel cantilever column systems.

The chapter is organized as follows:

E1. Ordinary Moment Frames
E2. Intermediate Moment Frames
E3. Special Moment Frames
E4. Special Truss Moment Frames
E5. Ordinary Cantilever Column Systems
E6. Special Cantilever Column Systems

E1. ORDINARY MOMENT FRAMES

1. Scope

Ordinary moment frames (OMF) of structural steel shall be designed in conformance with this section.

2. Basis of Design

OMF shall be designed to withstand minimal inelastic drift.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code*.

4. System Requirements

There are no additional requirements beyond the *Specification*.

5. Members

There are no limitations on width-thickness ratios of members for OMF, beyond those in the *Specification*. There are no requirements for lateral bracing of members or joints in OMF, beyond those in the *Specification*. There are no designated *protected zones* for OMF members. Structural steel beams in OMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.

Trusses shall be permitted as beams in OMF. The required strength of truss members shall be based on the maximum forces that can be delivered to the truss by the system, including overstrength and strain hardening. Truss connections shall be fully-restrained (FR) connections.

6. Connections

Beam-to-column connections are permitted to FR or partially-restrained (PR) moment connections in accordance with this section.

6a. Demand Critical Welds

Complete-joint-penetration welds of beam flanges to columns shall be *demand critical welds* in accordance with Section A3.4b.

User Note: Single-sided partial-joint-penetration groove welds and single-sided fillet welds should be avoided for resisting tensile forces in connections.

6b. Requirements for FR Moment Connections

FR moment connections that are part of the *seismic force resisting system* (SFRS) shall meet at least one of the following requirements:

(a) FR moment connections shall be designed for a *required flexural strength* that is equal to $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD), as appropriate, of the beam.

The *required shear strength*, V_u or V_a, as appropriate, of the connection shall be determined using the following quantity for the earthquake load effect E

$$E = 2[\ 1.1\ R_y\ M_p\ ] / L_{cf} \qquad \text{(E1-1)}$$

where

R_y = ratio of expected yield stress to the specified minimum yield stress, F_y

M_p = F_yZ, kip-in. (N-mm)

L_{cf} = clear length of beam, in. (mm)

(b) FR moment connections shall be designed for a required flexural strength and a required shear strength equal to the maximum moment and corresponding shear that can be transferred to the connection by the system, including the effects of material overstrength and strain hardening.

User Note: Factors that may limit the maximum moment and corresponding shear that can be transferred to the connection include:

(1) The strength of the columns, and

(2) The resistance of the foundations to uplift

For options (1) and (2) above, continuity plates should be provided as required by Sections J10.1, J10.2 and J10.3 of the *Specification*. The bending moment used to check for continuity plates should be the same bending moment used to design the beam-to-column connection; in other words, either $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD) or the maximum moment that can be transferred to the connection by the system.

(c) FR moment connections between wide flange beams and the flange of wide flange columns shall either meet the requirements of Section E2.6 or E3.6, or shall meet the following requirements:

(1) All welds at the beam-to-column connection shall meet the requirements of Chapter 3 of ANSI/AISC 358.

(2) Beam flanges shall be connected to column flanges using complete-joint-penetration (CJP) groove welds.

(3) The shape of weld access holes shall be in accordance with Section 6.9.1.2 of AWS D1.8. Weld access hole quality requirements shall be in accordance with Clause 6.9.2 of AWS D1.8.

(4) Continuity plates shall meet the requirements of Section 2.4.4 of ANSI/AISC 358, including Supplement No. 1. Continuity plates shall be at least as wide as the flanges of the wider connected beam.

Exception: The welded joints of the continuity plates to the column flanges are permitted to be complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds, or two-sided fillet welds. The *required strength* of these joints shall not be less than the *available strength* of the contact area of the plate with the column flange.

(5) The beam web shall be connected to the column flange using either a CJP groove weld extending between weld access holes, or using a bolted single plate shear connection designed for *required shear strength* per Equation E1-1.

User Note: For FR moment connections, panel zone shear strength should be checked according to Section J10.6 of the *Specification*. The required shear strength of the panel zone should be based on the beam end moments computed from the load combinations stipulated by the *applicable building code*, not including the amplified seismic load.

6c. Requirements for PR Moment Connections

PR moment connections are permitted when the following requirements are met:

(1) Such connections shall be designed for the maximum moment and shear from the applicable load combinations as described in Sections B2 and B3.

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

(3) The nominal flexural strength of the connection, $M_{n,PR}$ shall be no less than 50% of M_p of the connected beam.

Exception: For one-story structures, $M_{n,PR}$ shall be no less than 50% of M_p of the connected column.

(4) For PR moment connections, V_u or V_a as appropriate, shall be determined per Section E1.6b(a) with M_p in Equation E1-1 taken as $M_{n,PR}$.

E2. INTERMEDIATE MOMENT FRAMES

1. Scope

Intermediate moment frames (IMF) of structural steel shall be designed in conformance with this section.

2. Basis of Design

IMF shall be designed to provide limited inelastic drift capacity through flexural yielding of the IMF beams, , and columns, and shear yielding of the column panel zones. Design of connections of beams to columns, including panel zones and *continuity plates*, shall be based on connection tests that provide the performance required by Section E2.6b, and demonstrate this conformance as required by Section E2.6c.

User Note: Panel zone shear strength should be checked according to Section J10.6 of the *Specification*. The required shear strength of the panel zone should be based on the beam end moments computed from the load combinations stipulated by the *applicable building code*, not including the amplified seismic load.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code.*

4. System Requirements

4a. Stability Bracing of Beams

Beams shall be braced to meet the requirements for *moderately ductile members* in Section D1.2.

In addition, member 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 IMF. The placement of lateral bracing shall be consistent with that documented for a *prequalified connection* designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or in a program of qualification testing in accordance with Section J2.

The *required strength* of lateral bracing provided adjacent to plastic hinges shall be as required by Section D1.2c.

5. Members

5a. Width-Thickness Limitations

Beam and column members shall satisfy the requirements for *moderately ductile members*, as defined in Section D1.1b, unless otherwise qualified by tests.

5b. Beam Flanges

Abrupt changes in beam flange area shall not be permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width shall not be permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a *prequalified connection* designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or in a program of qualification testing in accordance with Section J2.

5c. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a *protected zone*, and shall meet the requirements of Section D1.3. The extent of the protected zone shall be as designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or as determined in a program of qualification testing in accordance with Section J2.

User Note: The plastic hinging zones at the ends of IMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection, per Section E2.6c. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.

5d. Beams

Structural steel beams in IMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.

6. Connections

6a. Demand Critical Welds

Unless otherwise designated by ANSI/AISC 358, including Supplement No. 1, or otherwise determined in a connection prequalification in accordance with Section J1, or as determined in a program of qualification testing in accordance with Section J2, complete-joint-penetration groove welds of beam flanges and beam webs to columns shall be *demand critical* welds as described in Section A3.4b.

User Note: For the designation of *demand critical welds*, standards such as ANSI/AISC 358 and tests addressing specific connections and joints should be used in lieu of the more general terms of these *Provisions*. Where these *Provisions* indicate that a particular weld is designated demand critical, but the more specific standard or test does not make such a designation, the more specific standard or test should govern. Likewise, these standards and tests may designate welds as demand critical that are not identified as such by these *Provisions*.

6b. Beam-to-Column Connection Requirements

Beam-to-column connections used in the SFRS shall satisfy the following requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.02 rad.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an *interstory drift angle* of 0.02 rad.

6c. Conformance Demonstration

Beam-to-column connections used in the SFRS shall satisfy the requirements of Section E2.6b by one of the following:

(a) Use of IMF connections designed in accordance with ANSI/AISC 358, including Supplement No. 1.

(b) Use of a connection prequalified for IMF in accordance with Section J1.

(c) Provision of qualifying cyclic test results in accordance with Section J2. 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 the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section J2.

(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 Section J2.

6d. Required Shear Strength

The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h \tag{E2-1}$$

where

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y

M_p= $F_y Z$ = nominal plastic flexural strength, kip-in. (N-mm)

L_h = distance between beam plastic hinge locations, in. (mm)

Exception: In lieu of Equation E2-1, the *required shear strength* of the connection shall be as specified in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section J1, or in a program of qualification testing in accordance with Section J2.

6e. Continuity Plates

Continuity plates shall be consistent with the *prequalified connection* designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or as determined in a program of qualification testing in accordance with Section J2.

6f. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where welds are used to make the splice, they shall be complete- joint-penetration groove welds.

When bolted column splices are used, they shall have a *required flexural strength* that is at least equal to $R_yF_yZ_x$ (LRFD) or $R_yF_yZ_x/1.5$ (ASD), as appropriate, of the smaller column. The *required shear strength* of column web splices shall be at least equal to $\Sigma M_{pc}/H$ (LRFD) or $\Sigma M_{pc}/1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Exception: The *required strength* of the column splice need not exceed that determined by a nonlinear analysis as specified in Chapter C considering appropriate stress concentration factors or fracture mechanics stress intensity factors.

E3. SPECIAL MOMENT FRAMES

1. Scope

Special moment frames (SMF) of structural steel shall be designed in conformance with this section.

2. Basis of Design

SMF shall be designed to provide significant inelastic drift capacity through flexural yielding of the SMF beams and limited yielding of column panel zones. Except where otherwise permitted in this section, columns shall be designed to be generally stronger than the fully yielded and strain-hardened beams or girders. Flexural yielding of columns at the base shall be permitted. Design of connections of beams to columns, including panel zones and *continuity plates*, shall be based on connection tests that provide the performance required by Section E3.6b, and demonstrate this conformance as required by Section E3.6c.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code*.

4. System Requirements

4a. Moment Ratio

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

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0 \qquad \text{(E3-1)}$$

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 columns (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 determine ΣM^*_{pc} as follows:

$$\Sigma M^*_{pc} = \Sigma Z_c(F_{yc}\text{-}P_{uc}/A_g) \text{ (LRFD)} \qquad \text{(E3-2a)}$$

or

$$\Sigma Z_c[(F_{yc}/1.5)\text{- } P_{ac}/A_g)] \text{ (ASD),} \qquad \text{(E3-2b)}$$

as appropriate.

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 moments in the beams at the intersection of the beam and column centerlines. ΣM^*_{pb} is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to determine ΣM^*_{pb} as follows:

$$\Sigma M^*_{pb} = \Sigma(1.1R_yF_{yb}Z_b+M_{uv}) \text{ (LRFD)} \qquad \text{(E3-3a)}$$

or

$$\Sigma M^*_{pb} = \Sigma[(1.1/1.5)R_yF_{yb}Z_b + M_{av}] \text{ (ASD)}, \quad \text{(E3-3b)}$$

as appropriate.

Alternatively, it is permitted to determine ΣM^*_{pb} consistent with a *prequalified connection* design as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section J1, or in a program of qualification testing in accordance with Section J2. When connections with reduced beam sections are used, it is permitted determine ΣM^*_{pb} as follows:

$$\Sigma M^*_{pb} = \Sigma(1.1R_yF_{yb}\, Z_{RBS} + M_{uv}) \text{ (LRFD)} \quad \text{(E3-4a)}$$

or

$$\Sigma M^*_{pb} = \Sigma[(1.1/1.5)R_yF_{yb}\, Z_{RBS} + M_{av}] \text{ (ASD)}, \quad \text{(E3-4b)}$$

as appropriate.

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

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

M_{av} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on ASD load combinations, kip-in. (N-mm)

M_{uv} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on LRFD load combinations, kip-in. (N-mm)

P_{ac} = required compressive strength using ASD load combinations, including the amplified seismic load, kips (N)

P_{uc} = required compressive strength using LRFD load combinations, including the amplified seismic load, kips (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_{RBS} = minimum plastic section modulus at the reduced beam section, in.^{3} (mm^3)

Exception: This requirement shall not apply if the conditions in (a) or (b) are satisfied.

(a) Columns with $P_{rc} < 0.3\ P_c$ for all *load combinations* other than those determined using the *amplified seismic load* that satisfy either of the following:

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

(ii) Columns where: (1) the sum of the *available shear strengths* of all *exempted columns* in the story is less than 20% of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction; and (2) the sum of the available shear strengths of all *exempted columns* on each moment frame column line within that story is less than 33% of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10% of the plan dimension perpendicular to the line of columns.

For design according to *Specification* Section B3.3 (LRFD),

P_c = $F_{yc}A_g$, kips (N)

P_{rc} = P_{uc}, required compressive strength, using *LRFD load combinations*, kips (N)

For design according to *Specification* Section B3.4 (ASD),

P_c = $F_{yc}A_g$ /1.5, kips (N)

P_{rc} = P_{ac}, required compressive strength, using *ASD load combinations*, kips (N)

(b) Columns in any story that has a ratio of available shear strength to *required shear strength* that is 50% greater than the story above.

4b. Stability Bracing of Beams

Beams shall be braced to meet the requirements for *highly ductile members* in Section D1.2b.

In addition, member 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. The placement of lateral bracing shall be consistent with that documented for a *prequalified connection* designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or in a program of qualification testing in accordance with Section J2.

The *required strength* of lateral bracing provided adjacent to plastic hinges shall be as required by Section D1.2c.

4c. Stability Bracing at Beam-to-Column Connections

(1) Braced Connections

When the webs of the beams and column are co-planar, and a column is shown to remain elastic outside of the panel zone, column flanges at beam-to-column connections shall require stability bracing only at the level of the top flanges of the beams. It shall be permitted to assume that the column remains elastic when the ratio calculated using Equation E3-1 is greater than 2.0.

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

(a) The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Stability bracing shall be permitted to be either direct or indirect.

User Note: Direct lateral support (bracing) of the column flange is achieved through use of member braces or other members, deck and slab, attached to the column flange at or near the desired bracing point to resist lateral buckling. Indirect lateral support refers to bracing that is achieved through the stiffness of members and connections that are not directly attached to the column flanges, but rather act through the column web or stiffener plates.

(b) Each column-flange member brace shall be designed for a *required strength* that is equal to 2% of the available beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf}$ / 1.5 (ASD), as appropriate.

(2) Unbraced Connections

A column containing a beam-to-column connection with no member bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent member braces as the column height for buckling transverse to the seismic frame and shall conform to *Specification* Chapter H, except that:

(1) The required column strength shall be determined from the appropriate *load combinations* in the *applicable building code*, except that E shall be permitted to be taken as the lesser of:

(i) the *amplified seismic load*

(ii) 125% of the frame *available strength* based upon either the beam *available flexural strength* or panel zone *available 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 E3.4c(1)(b) in addition to the second-order moment due to the resulting column flange lateral displacement.

5. Members

5a. Width-Thickness Limitations

Beam and column members shall meet the requirements of Section D1.1b and Table D1.1 for *highly ductile members*, unless otherwise qualified by tests.

5b. Beam Flanges

Abrupt changes in beam flange area shall not be permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width shall not be permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to the required interstory drift angle. The configuration shall be consistent with a *prequalified connection* designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or in a program of qualification testing in accordance with Section J2.

5c. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a *protected zone*, and shall meet the requirements of Section D1.3. The extent of the protected zone shall be as designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or as determined in a program of qualification testing in accordance with Section J2.

User Note: The plastic hinging zones at the ends of SMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection, per Section E3.6c. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.

5d. Beams

Structural steel beams in SMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.

6. Connections

6a. Demand Critical Welds

Unless otherwise designated by ANSI/AISC 358, including Supplement No. 1, or otherwise determined in a connection prequalification in accordance with Section J1, or as determined in a program of qualification testing in accordance with Section J2, complete-joint-penetration groove welds of beam flanges and beam webs to columns shall be *demand critical* welds as described in Section A3.4b.

User Note: For the designation of demand critical welds, standards such as ANSI/AISC 358 and tests addressing specific connections and joints should be used in lieu of the more general terms of these *Provisions*. Where these *Provisions* indicate that a particular weld is designated demand critical, but the more specific standard or test does not make such a designation, the more specific standard or test should govern. Likewise, these standards and tests may designate welds as demand critical that are not identified as such by these *Provisions*.

6b. Beam-to-Column Connection Requirements

Beam-to-column connections used in the *seismic force resisting system* (SFRS) shall satisfy the following two requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 rad.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 rad.

6c. Conformance Demonstration

Beam-to-column connections used in the SFRS shall satisfy the requirements of Section E3.6b by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358, including Supplement No. 1.

(b) Use of a connection prequalified for SMF in accordance with Section J1.

(c) Provision of qualifying cyclic test results in accordance with Section J2. Results of at least two cyclic connection tests shall be provided and shall be based on one of the following:

(i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section J2.

(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 Section J2.

6d. Required Shear Strength

The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h \qquad \text{(E3-5)}$$

where

R_y = ratio of the expected *yield stress* to the specified minimum *yield stress*, F_y

M_p = nominal plastic flexural strength, kip-in. (N-mm)

L_h = distance between plastic hinge locations, in. (mm)

Exception: In lieu of Equation E3-5, the *required shear strength* of the connection shall be as specified in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or in a program of qualification testing in accordance with Section J2.

6e. Panel Zone

(1) Required 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 or *prequalified connection*. As a minimum, the *required shear strength* 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 shall be $\phi_v R_v$ and the *allowable shear strength* shall be R_v/Ω_v where

$$\phi_v = 1.0 \text{ (LRFD)} \qquad \Omega_v = 1.50 \text{ (ASD)}$$

and the *nominal shear strength*, R_v, according to the limit state of shear yielding, is determined as specified in *Specification* Section J10.6.

(2) Panel Zone Thickness

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

$$t \geq (d_z + w_z)/90 \qquad \text{(E3-6)}$$

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 by using plug welds joining them, and dividing the plate to conform with Equation E3-1, the total panel zone thickness shall satisfy Equation E3-1. When plug welds are required, a minimum of four plug welds shall be provided.

(3) Panel Zone Doubler Plates

Doubler plates shall be welded to the column flanges to develop the available shear strength of the full doubler plate thickness, using either a complete-joint-penetration groove-welded or fillet-welded joint. 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.

User Note: When a doubler plate interferes with connecting continuity plates directly to the column web, the designer must provide a load path that satisfies the requirements of ANSI/AISC 358 Section 2.4.4b. This may be accomplished by sizing the doubler plate such that it is capable of developing the required strength of the continuity plate to column web connection. Alternatively, the doubler plate can stop inside the continuity plates. A similar load path must be provided when the web plate for a beam perpendicular to the column web connects to a doubler plate.

6f. Continuity Plates

Continuity plates shall be consistent with the prequalified connection designated in ANSI/AISC 358, including Supplement No. 1, or as otherwise determined in a connection prequalification in accordance with Section J1, or as determined in a program of qualification testing in accordance with Section J2.

6g. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds.

When bolted column splices are used, they shall have a *required flexural strength* that is at least equal to $R_yF_yZ_x$ (LRFD) or $R_yF_yZ_x$ /1.5 (ASD), as appropriate, of the smaller column. The *required shear strength* of column web splices shall be at least equal to $\Sigma M_{pc}/H$ (LRFD) or $\Sigma M_{pc}/1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

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 a nonlinear analysis as specified in Chapter C.

E4. SPECIAL TRUSS MOMENT FRAMES

1. Scope

Special truss moment frames (STMF) shall meet the requirements in this Section.

2. Basis of Design

STMF shall be designed to withstand significant inelastic deformation within a *special 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.

3. Analysis

3a. Special Segment

The *required vertical shear strength* of the special segment shall be calculated for the appropriate *load combinations* in the *applicable building code*.

3b. Non-Special Segment

The *required strength* of non-special segment members and connections shall be calculated based on the appropriate *load combinations* in the applicable building code, replacing the earthquake load term, E, with the lateral forces necessary to develop the *expected vertical shear strength* of the special segment acting at mid-length and defined in Section E4.5b. Second order effects at maximum design drift shall be included.

4. System Requirements

4a. Special Segment

Each horizontal truss that is part of the SFRS 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. Diagonal members within the special segment shall be made of rolled flat bars of identical sections. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a *required strength* equal to 0.25 times the nominal *tensile strength* of the diagonal member. Bolted connections shall not be used for diagonal members within the special segment.

Splicing of chord members shall not be permitted within the special segment, nor within one-half the panel length from the ends of the special segment.

The *required axial strength* of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03F_yA_g$ (LRFD) or $(0.03/1.5)F_yA_g$ (ASD), as appropriate.

4b. Stability Bracing of Trusses

Each flange of the chord members shall be laterally braced at the ends of the special segment. The *required strength* of the lateral brace shall be

$$P_u = 0.06\, R_y F_y A_f \text{ (LRFD)} \qquad \text{(E4-1a)}$$

or

$$P_a = (0.06/1.5)\, R_y F_y A_f \text{ (ASD)} \qquad \text{(E4-1b)}$$

where

A_f = gross area of the flange of the *special segment* chord member, in.^{2} (mm^2)

4c. Stability Bracing of Truss-to-Column Connections

The columns shall be laterally braced at the levels of top and bottom chords of the trusses connected to the columns. The lateral braces shall have required strength of

$$P_u = 0.02\, R_y P_{nc} \text{ (LRFD)} \qquad \text{(E4-2a)}$$

or

$$P_a = (0.02/1.5)\, R_y P_{nc} \text{ (ASD)} \qquad \text{(E4-2b)}$$

where

P_{nc} = nominal compressive strength of the chord member at the ends, kips (N)

4d. Stiffness of Stability Bracing

The required brace stiffness shall meet the provisions of Section 6.2 of Appendix 6 of the *Specification*, where

$$P_r = R_y P_{nc} \text{ (LRFD)} \qquad \text{(E4-3a)}$$

or

$$P_r = R_y P_{nc} / 1.5 \text{ (ASD)} \qquad \text{(E4-3b)}$$

5. Members

5a. Special Segment Members

The available shear strength of the *special segment* shall be calculated as the sum of the available shear strength of the chord members through flexure, and of the shear strength corresponding to the available tensile strength and 0.3 times the available compressive strength of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25% of the required vertical shear strength.

The *required axial strength* of the chord members shall not exceed 0.45 times ϕP_n (LRFD) or P_n / Ω (ASD), as appropriate,

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where

$$P_n = F_y A_g \qquad \text{(E4-4)}$$

5b. Expected Vertical Shear Strength of Special Segment

The *expected vertical shear strength* of the special segment, V_{ne} (LRFD) or $V_{ne}/1.5$ (ASD), as appropriate, at mid-length, shall be given as:

$$V_{ne} = \frac{3.60 R_y M_{nc}}{L_s} + 0.036 EI \frac{(L - L_s)}{L_s^3} + R_y (P_{nt} + 0.3 P_{nc}) \sin\alpha \qquad \text{(E4-4)}$$

where

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

E = modulus of elasticity of a chord member of the special segment, kip/in.^{2} (N/mm^2)

I = moment of inertia of a chord member of the special segment, in.^{4} (mm^4)

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

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

P_{nt} = nominal *tensile strength* of a diagonal member of the special segment, kips (N)

P_{nc} = nominal compressive strength of a diagonal member of the special segment, kips (N)

α = angle of diagonal members with the horizontal

5c. Width-Thickness Limitations

Chord members and diagonal web members within the special segment shall meet the requirements of Section D1.1b for highly ductile members. The width-thickness ratio of flat bar diagonal members shall not exceed 2.5.

5d. Built-Up Chord Members

Spacing of stitching for built-up chord members in the special segment shall not exceed $0.04Er_y/F_y$, where r_y is the radius of gyration of individual components about their weak axis.

5e. Protected Zones

The region at each end of a chord member within the special segment shall be designated as protected zone meeting the requirements of Section D1.3. The protected zone shall extend over a length equal to two times the depth of the chord member from the connection with the web members. Vertical and diagonal web members from end to end of the special segments shall be protected zones.

6. Connections

6a. Connections of Diagonal Web Members in the Special Segment

The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the *expected yield strength* of the web member, $R_yF_yA_g$ (LRFD) or $R_yF_yA_g$ / 1.5 (ASD), as appropriate.

6b. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds.

When bolted column splices are used, they shall have a *required flexural strength* that is at least equal to $R_yF_yZ_x$ (LRFD) or $R_yF_yZ_x$ /1.5 (ASD), as appropriate, of the smaller column. The *required shear strength* of column web splices shall be at least equal to $\Sigma M_{pc}/H$ (LRFD) or $\Sigma M_{pc}/1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

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 a nonlinear analysis as specified in Chapter C.

E5. ORDINARY CANTILEVER COLUMN SYSTEMS

1. Scope

Ordinary Cantilever Column Systems (OCCS) shall be designed in conformance with this section.

2. Basis of Design

OCCS shall be designed to provide minimal inelastic drift capacity through flexural yielding of the columns.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code.*

4. System Requirements

4a. Columns

Columns shall be designed using the load combinations including the amplified seismic load. The *required strength*, P_{uc}, shall not exceed $0.15P_y$.

4b. Stability Bracing of Columns

There are no additional requirements beyond the *Specification.*

5. Members

5a. Width-Thickness Limitations

There are no additional requirements beyond the *Specification.*

5b. Column Flanges

There are no additional requirements beyond the *Specification.*

5c. Protected Zones

There are no designated *protected zones.*

6. Connections

6a. Column Bases

There are no additional requirements beyond the *Specification.*

E6. SPECIAL CANTILEVER COLUMN SYSTEMS

1. Scope

Cantilever column systems (CCS) shall be designed in conformance with this section.

2. Basis of Design

CCS shall be designed to provide limited inelastic drift capacity through flexural yielding of the columns.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code.*

4. System Requirements

4a. Columns

Columns shall be designed using the load combinations including the amplified seismic load. The *required strength*, P_{uc}, shall not exceed $0.15P_y$.

4b. Stability Bracing of Columns

Columns shall be braced to meet the requirements for *moderately ductile members* in Section D1.2a.

5. Members

5a. Width-Thickness Limitations

Column members shall meet the requirements of Section D1.1b and Table D1.1 for *highly ductile members*.

5b. Column Flanges

Abrupt changes in column flange area are prohibited in the *protected zone* as designated in Section E3.5c.

5c. Protected Zones

The region at the base of the column subject to inelastic straining shall be designated as a *protected zone*, and shall meet the requirements of Section D1.3. The length of the protected zone shall be two times the column depth, unless otherwise substantiated by testing.

6. Connections

6a. Column Bases

Column bases shall be designed in accordance with Section D2.6.

CHAPTER F

BRACED-FRAME AND SHEAR-WALL SYSTEMS

This chapter provides the basis of design and the requirements for analysis, the system, members and connections for steel ordinary, special, eccentrically, and buckling-restrained braced frames, as well as special plate shear walls.

The chapter is organized as follows:

F1. Ordinary Concentrically Braced Frames
F2. Special Concentrically Braced Frames
F3. Eccentrically Braced Frames
F4. Buckling-Restrained Braced Frames
F5. Special Plate Shear Walls

F1. ORDINARY CONCENTRICALLY BRACED FRAMES

1. Scope

Ordinary concentrically braced frames (OCBF) shall be designed in conformance with this section.

In seismically isolated structures, OCBF above the isolation system shall meet the requirements of Sections F1.4b, F1.5, F1.6 and F1.7 and need not meet the requirements of Section F1.4a.

2. Basis of Design

This Section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities less than the beam depth are permitted if they are accounted for in the member design by determination of eccentric moments using the *amplified seismic load*. 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*.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code.*

4. System Requirements

4a. V-Braced and Inverted-V-Braced Frames

Beams in V-type and inverted V-type OCBF shall be continuous at brace connections away from the beam-column connection and shall meet the following requirements:

(1) The *required strength* shall be determined based on the *load combinations* of the *applicable building code* assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E, on the member shall be determined as follows:

(i) The forces in braces in tension shall be assumed to be the least of the following:

1. The expected strength of the brace, $R_yF_yA_g$

2. The load effect based upon the amplified seismic load

3. The maximum force that can be developed by the system

(ii) The forces in braces in compression shall be assumed to be equal to $0.3P_n$.

(2) As a minimum, one set of lateral braces is required at the point of intersection of the braces, unless the member has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

4b. K-Braced Frames

K-type braced frames are not permitted for OCBF.

5. Members

5a. Width-Thickness Limitations

Braces shall meet the requirements of Section D1.1a for *moderately ductile members*.

5b. Slenderness

Braces in V or inverted-V configurations shall have $Kl / r \leq 4\sqrt{E/F_y}$.

6. Connections

6a. Diagonal Brace Connections

The *required strength* of diagonal brace connections is the load effect based upon the *amplified seismic load.*

Exception: The required strength of the brace connection need not exceed the following:

(1) The maximum force that can be developed by the system

(2) In tension, the *expected yield strength* of the brace, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g /1.5$ (ASD), as appropriate

(3) In compression, 1.1 times the expected brace strength in compression. The expected brace strength in compression is permitted to be taken as the lesser of $R_y F_{cr} A_g$. and $F_E A_g$ where F_{cr} and F_E are determined from *Specification* Chapter E.

(4) When oversized holes are used, the required strength for the limit state of bolt slip need not exceed a *load effect* based upon using the *load combinations* stipulated by the applicable building code, not including the *amplified seismic load*.

User Note: In seismic design, slip is a strength limit state.

7. Ordinary Concentrically Braced Frames above Seismic Isolation Systems

7a. System Requirements

Beams in V-type and inverted V-type braced frames shall be continuous between columns.

7b. Members

Braces shall have $Kl/r \le 4\sqrt{E/F_y}$.

F2. SPECIAL CONCENTRICALLY BRACED FRAMES

1. Scope

Special concentrically braced frames (SCBF) shall be designed in conformance with this section. This Section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of system ductility: buckling and tension yielding of braces.

2. Basis of Design

SCBF shall be designed so that inelastic deformations under the design earthquake will occur primarily as brace buckling and yielding in tension.

3. Analysis

Columns, beams and connections in SCBF shall be designed to resist seismic forces from the *load combinations* in the *applicable building code* where the seismic load, E, on each element is the larger force determined from the following two analyses:

(i) An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension.

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

Braces shall be determined to be in compression or tension neglecting the effects of gravity loads. Analyses shall consider both directions of frame loading.

The expected brace strength in tension is $R_yF_yA_g$.

The expected brace strength in compression is permitted to be taken as the lesser of $R_yF_{cr}A_g$. and F_EA_g where F_{cr} and F_E are determined from the *Specification* Chapter E and the brace length is taken from brace end to brace end.

The expected post-buckling brace strength shall be taken as a maximum of 0.3 times the expected brace strength in compression.

User Note: Braces with a slenderness ratio of 200 (the maximum permitted by Section F2.5b) buckle elastically for permissible materials; the value of $0.3F_{cr}$ for such braces is 2.1 ksi. This value may be used in Section F2.3(ii) for braces of any slenderness and a liberal estimate of the required strength of framing members will be obtained. Alternatively, 0 ksi may also be used to simplify the analysis.

Exceptions:

(1) It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.

(2) The required strength of columns need not exceed the least of the following:

(a) The forces determined using load combinations stipulated by the applicable building code including the amplified seismic load, applied to a building frame model in which all compression braces have been removed.

(b) The forces corresponding to the resistance of the foundation to overturning uplift.

(c) Forces determined from nonlinear analysis as defined in Section C3.

4. System Requirements

4a. Lateral Force Distribution

Along any line of braces, braces shall be deployed in alternate directions such that, for either direction of force parallel to the braces, at least 30% but no more than 70% of the total horizontal force along that line is resisted by braces in tension, unless the *available strength* of each brace in compression is larger than the *required strength* resulting from the application of the appropriate *load combinations* stipulated by the *applicable building code* including the *amplified seismic load*. For the purposes of this provision, a line of braces is defined as a single line or parallel lines with a plan offset of 10% or less of the building dimension perpendicular to the line of braces.

4b. V- and Inverted V-Braced Frames

Beams that are intersected by braces away from beam-to-column connections shall comply with the following requirements:

(1) Beams shall be continuous between columns.

(2) Beams shall be braced to meet the requirements for *moderately ductile members* in Section D1.2a.

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

User Note: One method of demonstrating sufficient out-of-plane strength and stiffness of the beam is to apply the bracing force defined in Equation A-6-7 of Appendix 6 of the *Specification* to each flange so as to form a torsional couple; this loading should be in conjunction with the flexural forces defined in item (1) above. The stiffness of the beam (and its restraints) with respect to this torsional loading should be sufficient to satisfy Equation A-6-8 of the *Specification*.

4c. K-Braced Frames

K-type braced frames are not permitted for SCBF.

4d. Tension-Only Frames

Tension-only frames are not permitted in SCBF.

User Note: Tension-only braced frames are those in which the brace compression resistance is neglected in the design and the braces are designed for tension forces only.

5. Members

5a. Width-Thickness Limitations

Columns and braces shall meet the requirements of Section D1.1b for *highly ductile members*. Beams shall meet the requirements of Section D1.1b for *moderately ductile members*

5b. Diagonal Braces

Braces shall comply with the following requirements:

(1) Slenderness: Braces shall have a slenderness ratio $Kl/r \leq 200$.

(2) Built-up Braces: 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 sum of the available shear strengths of the stitches shall equal or exceed the available *tensile strength* of each element. The spacing of stitches shall be uniform. Not less than two stitches shall be used in a built-up member. Stitches shall not be located within the middle one-fourth of the clear brace length.

Exception: Where the buckling of braces about their critical bucking axis does not cause 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.

(3) The brace effective net area shall not be less than the brace gross area. Where reinforcement on braces is used the following requirements shall apply:

(i) The specified minimum yield strength of the reinforcement shall be at least the specified minimum yield strength of the brace.

(ii) The connections of the reinforcement to the brace shall have sufficient strength to develop the expected reinforcement strength on each side of a reduced section.

5c. Protected Zones

The protected zone of SCBF shall satisfy Section D1.3 and include the following:

(1) For braces, the center one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling.

(2) Elements that connect braces to beams and columns.

6. Connections

6a. Required Strength of Brace Connections

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

(1) Required Tensile Strength: The *required tensile strength* is the lesser of the following:

(a) The *expected yield strength*, in tension, of the brace, determined as $R_yF_yA_g$ (LRFD) or $R_yF_yA_g/1.5$ (ASD), as appropriate.

Exception: Braces need not comply with the requirements of Equation J4-1 and J4-2 of the *Specification* for this loading.

User Note: This exception applies to braces where the section is reduced or where the net section is effectively reduced due to shear lag. A typical case is a slotted HSS brace at the gusset plate connection. Section F2.5b requires braces with holes or slots to be reinforced such that the effective net area exceeds the gross area.

The brace strength used to check connection limit states, such as brace block shear, may be determined using expected material properties as permitted by Section A3.2.

(b) The maximum load effect, indicated by analysis, that can be transferred to the brace by the system.

When oversized (OVS) holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the *load combinations* stipulated by the *applicable building code*, including the *amplified seismic load*.

User Note: For other limit states the loadings of (i) and (ii) apply. In seismic design slip is a strength limit state.

(2) Required Compressive Strength: Brace connections shall be designed for a *required compressive strength* based on buckling limit states that is at least equal to 1.1 times the expected brace strength in compression (LRFD) or (1.1/1.5) times the expected brace strength in compression (ASD), as appropriate, where the expected brace strength in compression is as defined in Section F2.3(ii).

(3) Accommodation of Brace Buckling: Brace connections shall be designed to withstand the flexural forces or rotations imposed by brace buckling. Connections satisfying either of the following provisions are deemed to satisfy this requirement:

(i) Required Flexural Strength: Brace connections designed to withstand the flexural forces imposed by brace buckling shall have an available flexural strength of at least $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD), as appropriate, of the brace about the critical buckling axis.

(ii) Rotation Capacity: Brace connections designed to withstand the rotations imposed by brace buckling shall have sufficient rotation capacity to accommodate the required rotation at the *design story drift*. Inelastic rotation of the connection is permitted.

User Note: Accommodation of inelastic rotation is typically accomplished by means of a single gusset plate with the brace terminating before the line of restraint. The detailing requirements for such a connection are described in the commentary.

6b. Beam-to-Column Connections

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:

(1) The connection shall have sufficient rotation capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height; or

(2) The connection shall be designed to resist a moment equal to the lesser of the following:

(i) A moment corresponding to $1.1R_yF_yZ$ of the beam

(ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column

This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.

6c. Column Splices

In addition to meeting the requirements of Section D2.5, column splices in SCBF shall be designed to develop at least 50% of the lesser available flexural strength of the connected members.

The *required shear strength* shall be $\Sigma M_{pc} / H_c$ (LRFD) or $\Sigma M_{pc} / 1.5\ H_c$ (ASD), as appropriate,

where

ΣM_{pc} = sum of the nominal plastic flexural strengths, $F_{yc}Z_c$, of the columns above and below the splice, kip-in. (N-mm)

H_c = clear height of the column between beam connections, including a structural slab, if present., in. (mm)

6d. Demand Critical Welds

Groove welds at column splices shall be *demand critical welds* as described in Section A3.4b.

F3. ECCENTRICALLY BRACED FRAMES

1. Scope

Eccentrically braced frames (EBF) shall be designed in conformance with this section. Minor eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of system ductility: buckling and tension yielding of braces.

2. Basis of Design

EBF shall be designed so that inelastic deformations under the design earthquake will occur primarily in the links.

Where links connect directly to columns, the link-to-column connection shall provide the performance and shall meet the conformance demonstration requirements of Section F3.6a.

3. Analysis

The *required strength* of diagonal braces and their connections, beams outside links, and columns shall be based on the load combinations in the *applicable building code*. For load combinations that include earthquake effects, the earthquake effect E for columns, diagonal braces, connections and beams outside links shall be taken as the forces developed in the member assuming the forces at the end of the link connected to the member correspond to the *adjusted link shear strength*. The adjusted link shear strength shall be taken as R_y times the link nominal shear strength given in Section F3.5a(2) multiplied by 1.25 for I-shaped links and 1.4 for box links.

Exceptions:

(1) The earthquake effect, E, is permitted to be taken as 0.88 times the forces determined above for the design of the following members:

(a) Beams outside links; and
(b) Columns in frames of three or more stories of bracing.

(2) It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.

(3) The required strength of columns need not exceed the lesser of the following:

(a) Forces corresponding to the resistance of the foundation to overturning uplift.

(b) Forces as determined from nonlinear analysis as defined in Section C3.

The inelastic link rotation angle shall be determined from the inelastic story drift. Alternatively, the inelastic link rotation angle may be determined from nonlinear analysis as defined in Section C3.

User Note: The seismic load effect *E* used in the design of EBF members, such as the required axial strength used in the equations in Section F3.5, should be calculated from the analysis above.

4. System Requirements

4a. Link Rotation Angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the *design story drift*, Δ. The link rotation angle shall not exceed the following values:

(1) 0.08 rad for links of length $1.6M_p/V_p$ or less.

(2) 0.02 rad for links of length $2.6M_p/V_p$ or greater.

The value determined by linear interpolation between the above values for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

4b. Bracing of Link

Bracing shall be provided at both the top and bottom link flanges at the ends of the link for I-shaped sections. Bracing shall have an available strength and stiffness as required for expected plastic hinge locations by Section D1.2b for *highly ductile members*.

5. Members

5a. Links

Link beams subject to shear and flexure due to eccentricity between the intersections of brace centerlines and the beam centerline (or between the intersection of the brace and beam centerlines and the column centerline for links attached to columns) shall be provided. The link beam shall be considered to extend from brace connection to brace connection for center links and from brace connection to column face for link-to-column connections except as permitted by Section F3.6a.

(1) Limitations

Links shall be I-shaped cross-sections (rolled wide-flange sections or built-up sections), or built-up box sections. HSS sections shall not be used as links.

Links shall meet the requirements of Section D1.1b for *highly ductile members*.

Exception: Flanges of links with I-shaped sections with link lengths, $e \leq 1.6\ M_p/V_p$ are permitted to meet the requirements for *moderately ductile members*.

The web or webs of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

For links made of built-up cross-sections, complete-joint-penetration groove welds shall be used to connect the web (or webs) to the flanges.

Links of built-up box sections shall have a moment of inertia, I_y, about an axis in the plane of the EBF limited to $I_y > 0.67\ I_x$, where I_x is the moment of inertia about an axis perpendicular to the plane of the of the EBF.

(2) Shear Strength

The link *design shear strength*, $\phi_v V_n$, and the *allowable shear strength*, V_n/Ω_v, shall be the lower value obtained according to the *limit state*s of *shear yielding* in the web and *flexural yielding* in the gross section. For both limit states:

ϕ_v = 0.90 (LRFD) Ω_v = 1.67 (ASD)

For shear yielding:

$$V_n = V_p \tag{F3-1}$$

where

V_p = $0.6F_yA_{lw}$ for $P_r/P_c \leq 0.15$ (F3-2)

V_p = $0.6F_yA_{lw}\sqrt{1-\left(P_r/P_c\right)^2}$ for $P_r/P_c > 0.15$ (F3-3)

A_{lw} = $(d-2t_f)t_w$ for I-shaped link sections (F3-4)

= $2(d-2t_f)t_w$ for box link sections (F3-5)

P_r = P_u (LRFD) or P_a (ASD), as appropriate

P_u = required axial strength using LRFD load combinations, kips (N)

P_a = required axial strength using ASD load combinations, kips (N)

P_c = P_y (LRFD) or $P_{y/1.5}$ (ASD) as appropriate

P_y = nominal axial yield strength = F_yA_g (F3-6)

For flexural yielding:

$$V_n = 2M_p/e \quad \text{(F3-7)}$$

where

$M_p = F_y Z$ for $P_r/P_c \le 0.15$ (F3-8)

$M_p = F_y Z\left[\dfrac{1-P_r/P_c}{0.85}\right]$ for $P_r/P_c > 0.15$ (F3-9)

e = link length, in. (mm)

User Note: The requirements of Section F3.5a(2) and (3) have been reformatted from the 2005 *Seismic Provisions for Structural Steel Buildings* for clarity and simplicity. However, no change to the requirements is entailed in this reformatting.

(3) Link Length

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

When $\rho' \le 0.5$

$$e \le \frac{1.6M_p}{V_p} \quad \text{(F3-10)}$$

When $\rho' > 0.5$

$$e \le \frac{1.6M_p}{V_p}\left(1.15-0.3\rho'\right) \quad \text{(F3-11)}$$

where

$$\rho' = \frac{P_r/P_c}{V_r/V_c} \quad \text{(F3-12)}$$

V_r = V_u (LRFD) or V_a (ASD), as appropriate, kips (N)

V_u = required shear strength based on LRFD load combinations, kips (N)

V_a = required shear strength based on ASD load combinations, kips (N)

V_c = V_y (LRFD) or $V_y/1.5$ (ASD), as appropriate, kips (N)

V_y = nominal shear yield strength, kips (N)

= $0.6F_yA_{lw}$, (F3-13)

User Note: For links with low axial force there is no upper limit on link length. The limitations on link rotation angle in Section F3.4a result in a practical lower limit on link length.

(4) Link Stiffeners for I-shaped cross-sections

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

(i) 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 rad or ($52t_w$-d/5) for link rotation angles of 0.02 rad or less. Linear interpolation shall be used for values between 0.08 and 0.02 rad.

(ii) 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.

(iii) 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 (i) and (ii) above.

Intermediate web stiffeners are not required in links of length greater than $5M_p/V_p$.

Intermediate 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 $A_{st}F_y$ (LRFD) or $A_{st}F_y$ / 1.5 (ASD), as appropriate, where A_{st} is the horizontal cross-sectional area of the link stiffener. The *required strength* of fillet welds connecting the stiffener to the link flanges is $A_{st}F_y$ / 4 (LRFD) or $A_{st}F_y$ /4(1.5) (ASD).

(5) Link Stiffeners for box sections

Full-depth web stiffeners shall be provided on one side of each link web at the diagonal brace connection. These stiffeners may be welded to the outside or inside face of the link webs. These stiffeners shall each have a width not less than b/2, where b is the inside width of the box. These stiffeners shall each have a thickness not less than 0.75 t_w or ½ in. (13 mm), whichever is less.

Box links shall be provided with intermediate web stiffeners as follows:

(a) For links of length $1.6M_p/V_p$ or less and with web depth-to-thickness ratio, h/t_w, greater than or equal to $0.64\sqrt{E/F_y}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20t_w$-(d-$2t_f$)/8.

(b) For links of length 1.6 M_p/V_p or less and with web depth-to-thickness ratio, h/t_w less than $0.64\sqrt{E/F_y}$, no intermediate web stiffeners are required.

(c) For links of length greater than $1.6M_p/V_p$, no intermediate web stiffeners are required.

Intermediate web stiffeners shall be full depth, and may be welded to the outside or inside face of the link webs.

The *required strength* of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y$ / 1.5 (ASD), as appropriate, where A_{st} is the horizontal cross-sectional area of the link stiffener.

User Note: Stiffeners of box links need not be welded to link flanges.

5b. Diagonal Braces

Brace members shall satisfy width-thickness limitations in Section D1.1b for *moderately ductile members*.

5c. Beam Outside Link

Where the beam outside of the link is a different section from the link, the beam shall satisfy the width-thickness limitations in Section D1.1b for *moderately ductile members.*

User Note: The diagonal brace and beam segment outside of the link are intended to remain essentially elastic under the forces generated by the fully yielded and strain hardened link. Both the diagonal brace and beam segment outside of the link are typically subject to a combination of large axial force and bending moment, and therefore should be treated as beam-columns in design, where the available strength is defined by Chapter H of the *Specification*.

Where the beam outside the link is the same member as the link, its strength may be determined using expected material properties as permitted by Section A3.2.

5d. Columns

Column members shall satisfy width-thickness limitations in Section D1.1b for *highly ductile members.*

5e. Protected Zones

Links in EBFs are a protected zone, and shall satisfy the requirements of Section D1.3.

6. Connections

6a. Link-to-Column Connections

(1) Requirements

Link-to-column connections shall be fully restrained (FR) moment connections and shall satisfy the following requirements:

(i) The connection shall be capable of sustaining the link rotation angle specified in Section F3.4a.

(ii) The shear resistance of the connection, measured at the required link rotation angle, shall be at least equal to the nominal shear strength of the link, V_n, as defined in Section F3.5a(2).

(iii) The flexural resistance of the connection, measured at the required link rotation angle, shall be at least equal to the moment corresponding to the nominal shear strength of the link, V_n, as defined in Section F3.5a(2).

(2) Conformance Demonstration

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

(a) Use a connection *prequalified* for EBF in accordance with Section J1.

User Note: There are no prequalified link-to-column connections.

(b) Provide qualifying cyclic test results in accordance with Section J2. 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 representative of project conditions, within the limits specified in Section J2.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection material properties, within the limits specified in Section J2.

Exception: Cyclic testing of the reinforced connection is not required if the following conditions are met:

(1) Reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length.

(2) The *available strength* of the reinforced section and the connection equals or exceeds the *required strength* calculated based upon *adjusted link shear strength* as described in Section F3.3.

(3) The link length (taken as the beam segment from the end of the reinforcement to the brace connection) does not exceed $1.6M_p/V_p$.

(4) Full depth stiffeners as required in Section F3.5a(4) are placed at the link-to-reinforcement interface.

6b. Diagonal Brace Connections

When oversized (OVS) holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the *load* *combinations* stipulated by the *applicable building code*, including the *amplified seismic load*.

Connections of braces designed to resist a portion of the link end moment shall be designed as fully restrained.

6c. Beam-to-Column Connections

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:

(1) The connection shall have sufficient capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height; or

(2) The connection shall be designed to resist a moment simultaneous with the required strength of the brace connection, equal to the lesser of the following:

(i) A moment corresponding to $1.1R_yF_yZ$ of the beam.

(ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column.

This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.

6d. Column Splices

In addition to meeting the requirements of Section D2.5, column splices in EBF shall be designed to develop at least 50% of the lesser available flexural strength of the connected members.

The *required shear strength* shall be $\Sigma M_{pc} / H_c$ (LRFD) or $\Sigma M_{pc} / 1.5\ H_c$ (ASD), as appropriate,

where

ΣM_{pc} = the sum of the nominal plastic flexural strengths $F_{yc}Z_c$ of the columns above and below the splice, kip-in. (N-mm)

H_c = is the clear height of the column between beam connections, including a structural slab, if present. in. (mm)

6e. Demand Critical Welds

The following welds are *demand critical* welds, and shall satisfy the requirements of Section A3.4b:

(1) Welds attaching the link flanges and the link web to the column where links connect to columns,

(2) Welds connecting the webs to the flanges in built-up link beams.

(3) Groove welds at column splices.

F4. BUCKLING-RESTRAINED BRACED FRAMES

1. Scope

Buckling-restrained braced frames (BRBF) shall be designed in conformance with this section. Minor eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of system ductility: compression and tension yielding of braces.

2. Basis of Design

The analysis of BRBF shall be as required by Section F4.3. Braces shall provide the performance required by Section F4.2a, and demonstrate this conformance as required by Section F4.5a(3). Braces shall be designed, tested, and detailed to accommodate expected deformations. Expected deformations are those corresponding to a story drift of at least 2% of the story height or two times the design story drift, whichever is larger, in addition to brace deformations resulting from deformation of the frame due to gravity loading..

BRBF shall be designed so that inelastic deformations under the design earthquake will occur primarily as brace yielding in tension and compression.

2a. Brace Strength

The adjusted brace strength shall be established on the basis of testing as described in this section.

Where required by these *Provisions*, brace connections and adjoining members shall be designed to resist forces calculated based on the adjusted brace strength.

The adjusted brace strength in compression shall be $\beta\omega R_y P_{ysc}$, where

β = compression strength adjustment factor

ω = strain hardening adjustment factor

P_{ysc}= axial yield strength of steel core, ksi (MPa)

The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$.

Exception: The factor R_y need not be applied if P_{ysc} is established using yield stress determined from a coupon test.

The compression strength adjustment factor, β, shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Section J3.4c for the expected deformations. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0.

The strain hardening adjustment factor, ω, shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Section J3.4c (for the expected deformations) to P_{ysc} of the test specimen. The larger value of ω from the two required qualification tests shall be used. Where the tested *steel core* material does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

3. Analysis

Buckling-restrained braces shall not be considered as resisting gravity forces.

Columns, beams, and connections in BRBF shall be designed to resist seismic forces from the *load combinations* in the *applicable building code* where the seismic load E is determined from an analysis in which all braces are assumed to resist forces corresponding to their adjusted strength in compression or in tension.

Braces shall be determined to be in compression or tension neglecting the effects of gravity loads. Analyses shall consider both directions of frame loading.

The adjusted brace strength in tension shall be as given in Section F4.2a.

Exceptions:

(1) It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.

(2) The required strength of columns need not exceed the lesser of the following:

(i) The forces corresponding to the resistance of the foundation to overturning uplift.

(ii) Forces as determined from nonlinear analysis as defined in Section C3.

The brace deformation shall be determined from the inelastic story drift and shall include the effects of beam vertical flexibility. Alternatively, the brace deformation may be determined from nonlinear analysis as defined in Section C3.

4. System Requirements

4a. V- and Inverted V-Braced Frames

V-type and inverted-V-type braced frames shall meet the following requirements:

(1) The *required strength* of beams intersected by braces, their connections, and supporting members shall be determined based on the *load combinations* of the *applicable building code* assuming that the braces provide no support for dead and live loads. For *load combinations* that include earthquake effects, the vertical and horizontal earthquake effect, *E*, on the beam shall be determined from the adjusted brace strengths in tension and compression.

(2) Beams shall be continuous between columns. Beams shall be braced to meet the requirements for *moderately ductile members* in Section D1.1. As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) braces, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

User Note: The beam has sufficient out-of-plane strength and stiffness if the beam bent in the horizontal plane meets the required brace strength and required brace stiffness for column nodal bracing as prescribed in the *Specification*. P_u may be taken as the required compressive strength of the brace.

For purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam under the loading defined in Section F4.4a(1).

4b. K-Braced Frames

K-type braced frames are not permitted for BRBF.

4c. Bracing Connections at Beam-to-Column Connections

Bracing connections at beam-to-column connections shall be designed to withstand an out-of-plane earthquake force E equal to 6% of the adjusted diagonal brace compression strength The required bracing stiffness shall meet the provisions of Equation A-6-4 of Appendix 6 of the *Specification* where P_r is the adjusted diagonal brace compression strength.

5. Members

5a. Diagonal Braces

(1) Assembly

Braces shall be composed of a structural *steel core* and a system that restrains the *steel core* from buckling.

(i) Steel Core

Plates used in the *steel core* that are 2 in. (50 mm) thick or greater shall satisfy the minimum notch toughness requirements of Section A3.3.

Splices in the *steel core* are not permitted.

(ii) Buckling-Restraining System

The buckling-restraining system shall consist of the casing for the *steel core*. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the *steel core* for the expected deformations.

User Note: Conformance to this provision is demonstrated by means of testing as described in Section F4.2b.

(2) Available Strength

The *steel core* shall be designed to resist the entire axial force in the brace.

The brace design axial strength, ϕP_{ysc} (LRFD), and the brace allowable axial strength, P_{ysc}/Ω (ASD), in tension and compression, according to the limit state of yielding, shall be determined as follows:

$$P_{ysc} = F_{ysc} A_{sc} \tag{F4-1}$$

$\phi = 0.90$ (LRFD) $\qquad \Omega = 1.67$ (ASD)

where

F_{ysc} = specified minimum yield stress of the *steel core*, or actual yield stress of the *steel core* as determined from a coupon test, ksi (MPa)

A_{sc} = net area of *steel core*, in.^{2} (mm^2)

User Note: Load effects calculated based on adjusted brace strengths should not be amplified by the overstrength factor, Ω_o.

(3) Conformance Demonstration

The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Section J3. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace subassemblage that includes brace connection rotational demands complying with Section J3.2 and the other shall be either a uniaxial or a subassemblage test complying with Section J3.3. Both test types shall be based upon one of the following:

(a) Tests reported in research or documented tests performed for other projects.

(b) Tests that are conducted specifically for the project.

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of *steel core* and buckling-restraining system sizes. Tests are permitted to qualify a design when the provisions of Section J3 are met.

5b. Width-Thickness Limitations of Beams and Columns

Beam and column members shall meet the requirements of Section D1.1b for *highly ductile members*.

5c. Protected Zones

The protected zone shall include the *steel core* of braces and elements that connect the *steel core* to beams and columns, and shall satisfy the requirements of Section D1.3.

6. Connections

6a. Diagonal Brace Connections

(1) Required Strength

The *required strength* of brace connections in tension and compression (including beam-to-column connections if part of the braced-frame system) shall be 1.1 times the adjusted brace strength in compression (LRFD) or 1.1/1.5 times the adjusted brace strength in compression (ASD) where the adjusted brace strength is as defined in Section F4.2a.

When oversized (OVS) holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the *load combinations* stipulated by the *applicable building code*, including the *amplified seismic load*.

(2) Gusset-Plate Requirements

The design of connections shall include considerations of local and overall buckling. Lateral bracing consistent with that used in the tests upon which the design is based is required.

User Note: This provision may be met by designing the gusset plate for a transverse force consistent with transverse bracing forces determined from testing, by adding a stiffener to it to resist this force, or by providing a brace to the gusset plate. Where the supporting tests did not include transverse bracing, no such bracing is required. Any attachment of bracing to the steel core must be included in the qualification testing.

6b. Column Splices

In addition to meeting the requirements of Section D2.5, column splices in BRBF shall be designed to develop at least 50% of the lesser available flexural strength of the connected members, determined based on the limit state of yielding. The *required shear strength* shall be $\Sigma M_{pc} / H_c$ (LRFD) or $\Sigma M_{pc} / 1.5H_c$ (ASD), as appropriate,

where

ΣM_{pc} = sum of the nominal plastic flexural strengths, Z_cF_{yc}, of the columns above and below the splice, kip-in. (N-mm)

H_c = clear height of the column between beam connections, including a structural slab, if present, in. (mm)

6c. Demand Critical Welds

Groove welds of column splices shall be *demand critical welds* as described in Section A3.4b.

6d. Beam-to-Column Connections

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:

(a) The connection shall have sufficient capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height; or

(b) The connection shall be designed to resist a moment equal to the lesser of the following:

(i) A moment corresponding to $1.1R_yF_yZ$ of the beam.

(ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column.

This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.

F5. SPECIAL PLATE SHEAR WALLS

1. Scope

Special plate shear walls (SPSW) shall be designed in conformance with this section.

2. Basis of Design

SPSW are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the *design earthquake*. SPSW shall be designed so that inelastic deformations under the design earthquake will occur as web plate yielding and as plastic-hinge formation in the ends of horizontal boundary elements (HBEs).

3. Analysis

The webs of SPSW shall not be considered as resisting gravity forces.

HBEs, VBEs and connections in SPSW shall be designed to resist seismic forces from the *load combinations* in the *applicable building code* where the seismic load E is determined from the following analysis:

An analysis in which all webs are assumed to resist forces corresponding to their expected strength in tension at an angle α as determined in Section F5.5a and HBE are resisting flexural forces corresponding to flexural forces at each end equal to $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD). Webs shall be determined to be in tension neglecting the effects of gravity loads.

The expected web yield stress shall be taken as R_yF_y. Where perforated walls are used, the effective expected tension stress is as defined in Section F5.7a.

User Note: Shear forces per Equation E1-1 must be included in this analysis. Designers should be aware that in some cases forces from the analysis in the applicable building code will govern the design of HBE.

User Note: Shear forces in beams and columns are likely to be high and shear yielding may be a governing limit state.

4. System Requirements

4a. Stiffness of Boundary Elements

The vertical boundary elements (VBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c, not less than $0.0031\, t_w\, h^4 / L$. The horizontal boundary elements (HBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, not less than $0.0031\, L^4 / h$ times the difference in web plate thicknesses above and below,

where

t_w = thickness of the web, in. (mm)

h = distance between HBE centerlines, in. (mm)

I_b = moment of inertia of a HBE taken perpendicular to the direction of the web plate line, in.^{4} (mm^4)

I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, in.^{4} (mm^4)

L = distance between VBE centerlines, in. (mm)

4b. HBE-to-VBE Connection Moment Ratio

The beam-column moment ratio provisions in Section E3.4a shall be met for all HBE/VBE intersections without consideration of the effects of the webs.

4c. Bracing

HBE shall be braced to meet the requirements for *moderately ductile members* in Section D1.2a.

4d. Openings in Webs

Openings in webs shall be bounded on all sides by intermediate boundary elements extending the full width and height of the panel respectively, unless otherwise justified by testing and analysis or permitted by Section F5.7.

5. Members

5a. Webs

The panel design shear strength, ϕV_n (LRFD), and the *allowable shear strength*, V_n/Ω (ASD), according to the limit state of shear yielding, shall be determined as follows:

$$V_n = 0.42\, F_y\, t_w\, L_{cf} \sin 2\alpha \qquad \text{(F5-1)}$$

$$\phi = 0.90 \quad \text{(LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where

t_w = thickness of the web, in. (mm)

L_{cf} = clear distance between column flanges, in. (mm)

α = angle of web yielding in radians, as measured relative to the vertical. The angle of inclination, α, is permitted to be taken as 40°, or may be calculated as follows:

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w L}{2A_c}}{1 + t_w h\left(\dfrac{1}{A_b} + \dfrac{h^3}{360 I_c L}\right)} \qquad \text{(F5-2)}$$

h = distance between HBE centerlines, in. (mm)

A_b = cross-sectional area of a HBE, in.^{2} (mm^2)

A_c = cross-sectional area of a VBE, in.^{2} (mm^2)

I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, in.^{4} (mm^4)

L = distance between VBE centerlines, in. (mm)

5b. Width-Thickness Limitations

HBE, VBE and intermediate boundary elements shall meet the requirements of Section D1.1b for *highly ductile members*.

5c. Protected Zone

The protected zone of SPSW shall satisfy Section D1.3 and include the following:

(1) The webs of SPSW.

(2) Elements that connect webs to HBEs and VBEs.

(3) The plastic hinging zones at each end of HBEs, over a region ranging from the face of the column to one beam depth beyond the face of the column, or as otherwise specified in Section E3.5c.

6. Connections

6a. Connections of Webs to Boundary Elements

The *required strength* of web connections to the surrounding HBE and VBE shall equal the *expected yield strength*, in tension, of the web calculated at an angle, α.

6b. HBE-to-VBE Connections

HBE-to-VBE connections shall satisfy the requirements of Section E1.6.

(1) Required Strength

The *required shear strength*, of a HBE-to-VBE connection shall be determined using a earthquake load effect, E, equal to the shear calculated from Equation E1-1 together with the shear resulting from the *expected yield strength* in tension of the webs yielding at an angle, α.

(2) Panel Zones

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section E3.6e.

6c. Column Splices

In addition to meeting the requirements of Section D2.5, column splices in SPSW shall be designed to develop at least 50% of the lesser available flexural strength of the connected members, determined based on the limit state of yielding. Groove welds of column splices shall be *demand critical* welds as described in Section A3.4b.

7. Perforated Webs

7a. Regular Layout of Circular Perforations

A perforated plate conforming to this section is permitted to be used as the web of a SPSW. Perforated webs shall have a regular pattern of holes of uniform diameter spaced evenly over the entire web-plate area in an array pattern so that holes align diagonally at a uniform angle to vertical. Edges of openings shall have a surface roughness of 500 micro-inches (13 microns) or less.

(1) Strength

The panel design shear strength, ϕV_n (LRFD), and the *allowable shear strength*, V_n/Ω (ASD), according to the limit state of shear yielding, shall be determined as follows for perforated webs:

$$V_n = 0.42\, F_y\, t_w\, L_{cf}\,(1 - 0.7\, D/S_{diag}) \qquad \text{(F5-3)}$$

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

where

D = the diameter of the holes, in. (mm)
S_{diag} = the shortest center-to-center distance between the holes, in. (mm)

2) Spacing

The spacing, S_{diag}, shall be at least $1.67D$.

The distance between the first holes and web connections to the HBEs and VBEs shall be at least D, but not exceeding $(D{+}0.7S_{diag})$.

(3) Stiffness

The stiffness of such regularly perforated infill plates shall be calculated using an effective web-plate thickness, t_{eff}, is given by:

$$t_{eff} = \frac{1-\frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)}{\left(1-\frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)\right)\left(1-\frac{N_r D \sin\alpha}{H_c}\right)} t_w \qquad \text{(F5-4)}$$

where

H_c = clear column (and web-plate) height between beam flanges, in. (mm)

t_w = web-plate thickness, in. (mm)

N_r = the number or horizontal rows of perforations

α = angle of the shortest center-to-center lines in the opening array to vertical, degrees

(4) Effective expected tension stress

The effective expected tension stress to be used in place of the effective tension stress for analysis per Section F5.3 is $R_y F_y (1 - 0.7\, D/S_{diag})$.

7b. Reinforced Corner Cut-out

Quarter-circular cut-outs are permitted at the corners of the webs provided that the webs are connected to a reinforcement arching plate following the edge of the cut-outs. The plates shall be designed to allow development of the full strength of the solid web and maintain its resistance when subjected to deformations corresponding to the design story drift. This is deemed to be achieved if the following conditions are met.

(a) Design for tension

The arching plate shall have the *available strength* to resist the axial tension force resulting from web-plate tension in the absence of other forces.

$$P_u = \frac{R_y F_y t_w R^2}{4e} \quad \text{(LRFD)} \qquad \text{(F5-5a)}$$

or

$$P_a = \frac{R_y F_y t_w R^2 / 1.5}{4e} \quad \text{(ASD), as appropriate} \qquad \text{(F5-5b)}$$

where

R = the radius of the cut-out, in. (mm)

$$e = R\left(1-\sqrt{2}/2\right), \text{ in. (mm)} \qquad \text{(F5-6)}$$

HBEs and VBEs shall be designed to resist the tension axial forces acting at the end of the arching reinforcement.

(b) Design for beam-to-column connection forces

The arching plate shall have the available strength to resist the combined effects of axial force and moment resulting from connection deformation in the absence of other forces. These forces are:

$$P_u = \frac{15\,EI_y}{4\,e^2}\frac{\Delta}{h} \quad \text{(LRFD)} \qquad \text{(F5-7a)}$$

or

$$P_a = \frac{15\,EI_y}{1.5\left(4\,e^2\right)}\left(\frac{\Delta}{h}\right) \quad \text{(ASD), as appropriate} \qquad \text{(F5-7b)}$$

$$M_{uy} = \frac{P_u R}{2} \quad \text{(LRFD)} \qquad \text{(F5-8a)}$$

or

$$M_{ay} = \frac{P_a R}{2} \quad \text{(ASD), as appropriate} \qquad \text{(F5-8b)}$$

where

E = modulus of elasticity, ksi (MPa)
I_y = moment of inertia of the plate, in.^{4} (mm^4)
h =the story height, in. (mm)
Δ =the design story drift, in. (mm)

CHAPTER G

COMPOSITE MOMENT FRAME SYSTEMS

G1. COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)

1. Scope

Composite ordinary moment frames (C-OMF) shall be designed in conformance with this section. This Section is applicable to moment frames with fully-restrained (FR) connections that consist of either composite or reinforced concrete columns and structural steel, concrete-encased composite, or *composite beams*.

2. Basis of Design

C-OMF are expected to withstand minimal inelastic drift.

User Note: Composite ordinary moment frames, comparable to reinforced concrete ordinary moment frames, are only permitted in Seismic Design Categories B or below in ASCE 7. This is in contrast to steel ordinary moment frames, which are permitted in higher Seismic Design Categories. The design requirements are commensurate with providing minimal ductility in the members and connections.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code*.

4. System Requirements

There are no additional requirements beyond those in the *Specification* and the *applicable building code*.

5. Members

There are no additional requirements for steel or composite members beyond those in the *Specification*. Reinforced concrete columns shall meet the requirements of ACI 318, excluding Chapter 21.

6. Connections

Connections shall be fully-restrained (FR). Connections shall be designed for the applicable load combinations as described in Sections B2 and B3..

Beam-to-column connection design strengths shall be determined according to the *Specification* and Section D2.7.

G2. COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)

1. Scope

Composite intermediate moment frames (C-IMF) shall be designed in conformance with this section. This Section is applicable to moment frames with fully-restrained (FR) connections that consist of either composite or reinforced concrete columns and structural steel, concrete-encased composite or *composite beams*.

2. Basis of Design

C-IMF shall be designed to provide limited inelastic drift capacity through flexural yielding of the C-IMF beams, column panel zones, and columns.

User Note: Composite intermediate moment frames, comparable to reinforced concrete intermediate moment frames, are only permitted in Seismic Design Categories C or below in SEI/ASCE 7. This is in contrast to steel intermediate moment frames, which are permitted in higher Seismic Design Categories. The design requirements are commensurate with providing limited ductility in the members and connections.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code.*

4. System Requirements

4a. Stability Bracing of Beams

Beams shall be braced to meet the requirements for *moderately ductile members* in Section D1.2a.

In addition, member braces shall be placed near concentrated loads, changes in cross-section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the C-IMF.

The *required strength* of stability bracing provided adjacent to plastic hinges shall be as required by Section D1.2c.

5. Members

5a. Detailing Requirements

Steel and composite members shall meet the requirements of Sections D1.2, D1.4b and D1.4c for *moderately ductile members*.

5b. Columns

Steel, composite, and reinforced concrete columns shall meet the requirements of Section D1.4a.

5c. Beam Flanges

Abrupt changes in the beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is not permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges.

5d. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall meet the requirements of Section D1.3.

User Note: The plastic hinge zones at the ends of C-IMF beams should be treated as *protected zones*. In general, the protected zone will extend from the face of the composite column to one-half of the beam depth beyond the plastic hinge point.

6. Connections

Connections shall be fully-restrained (FR) and shall meet the requirements of Section D2 and this section.

User Note: All subsections of Section D2 are relevant for C-IMF.

6a. Demand Critical Welds

Complete-joint-penetration groove welds of beam flanges, diaphragm plates, shear plates, and beam webs to columns shall be *demand critical* welds as described in Section A3.4b.

6b. Beam-to-Column Connection Requirements

Beam-to-composite column connections used in the SFRS shall satisfy the following requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.02 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam

at an *interstory drift angle* of 0.02 rad, where M_p is defined as the nominal flexural strength of the steel, concrete-encased or composite beams and shall meet the requirements of *Specification* Chapter I.

6c. Required Shear Strength

The *required shear strength* of the connection, V_u, shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1M_{p,exp}]/L_h \tag{G2-1}$$

where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-encased or composite beams. For concrete-encased or composite beams, $M_{p,exp}$ shall be calculated using the plastic stress distribution or the strain compatibility method. Appropriate R_y factors shall be used for different elements of the cross-section while establishing section force equilibrium and calculating the flexural strength. L_h shall be equal to the distance between beam plastic hinge locations, in. (mm).

User Note: For steel beams, $M_{p,exp}$ in Equation G2-1 may be taken as R_yM_p of the beam.

6d. Continuity Plates and Diaphragms

Connection diaphragm plates are permitted for filled composite columns both external to the column and internal to the column.

Where diaphragm plates are used, the thickness of the plates shall be at least the thickness of the beam flange.

The diaphragm plates shall be welded around the full perimeter of the column using either complete-joint-penetration welds or two sided fillet welds. The *required strength* of these joints shall not be less than the *available strength* of the contact area of the plate with the column sides.

Internal diaphragms shall have circular openings sufficient for placing the concrete.

6e. Column Splices

In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. When column splices are not made with groove welds, they shall have a *required flexural strength* that is at least equal to the moment capacity M^*_{pcc} of the smaller composite column. The *required shear strength* of column web splices shall be at least equal to $\Sigma M^*_{pcc}/H$, where ΣM^*_{pcc} is the sum of the nominal flexural strengths of the composite columns above and below the splice. M^*_{pcc} is defined as the nominal flexural strength of the composite or reinforced concrete column. For composite columns, the nominal flexural strength shall meet the requirements of *Specification* Chapter I with consideration of the required axial strength, P_{rc}. For reinforced concrete columns, the nominal flexural strength shall be calculated based on the provisions of ACI 318 with consideration of the required axial strength, P_{rc}.

G3. COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

1. Scope

Composite special moment frames (C-SMF) shall be designed in conformance with this section. This Section is applicable to moment frames with fully-restrained (FR) connections that consist of either composite or reinforced concrete columns and either structural steel or concrete-encased composite, or *composite beams*.

2. Basis of Design

C-SMF shall be designed to provide significant inelastic drift capacity through flexural yielding of the C-SMF beams and limited yielding of the column panel zones. Except where otherwise permitted in this section, columns shall be designed to remain essentially elastic when subjected to forces associated with fully yielded and strain hardened beams or girders. Flexural yielding columns at the base shall be permitted.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code*.

4. System Requirements

4a. Moment Ratio

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

$$\frac{\sum M^*_{pcc}}{\sum M^*_{p,\exp}} > 1.0 \qquad \text{(G3-1)}$$

where

ΣM^*_{pcc} = the sum of the nominal flexural strengths of the composite columns above and below the joint at the intersection of the beam and column centerlines. ΣM^*_{pcc} is determined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column, where M^*_{pcc} is defined in Section G2.6e. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

$\Sigma M^*_{p,exp}$ = the sum of the expected flexural strengths of the steel beams or concrete-encased composite beams at the intersection of the beam and column centerlines. $\Sigma M^*_{p,exp}$ is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\Sigma M^*_{p,exp} = \Sigma(1.1M_{p,exp}+M_{uv})$, where $M_{p,exp}$ is calculated as specified in Section G2.6c.

M_{uv} = moment due to shear amplification from the location of the plastic hinge to the column centerline

Exception: The exceptions of Section E3.4a shall apply with the following modifications.

(1) For exception (1), the force limit in Section E3.4a shall be $P_{rc} < 0.1P_c$.

(2) *Composite columns* exempted from the requirement of Equation G3-1 shall have transverse reinforcement that meets the requirements of Section D1.4b(2)(ii).

4b. Stability Bracing of Beams

Beams shall be braced to meet the requirements for *highly ductile members* in Section D1.2b.

In addition, stability bracing 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 C-SMF.

The *required strength* of stability bracing provided adjacent to plastic hinges shall be as required by Section D1.2c.

4c. Stability Bracing at Beam-to-Column Connections

Composite columns with unbraced connections shall satisfy the requirements of Section E3.4c(2).

5. Members

5a. Detailing Requirements

Steel and composite members shall meet the requirements of Sections D1.1, D1.4b and D1.4c for *highly ductile members*.

Exception: Reinforced concrete-encased beams shall satisfy the requirements for Section D1.1 for *moderately ductile members* if the reinforced concrete cover is 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 of ACI 318 Section 21.5.3.

Concrete-encased composite beams that are part of C-SMF shall also meet the following requirements. The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed the following requirement

$$Y_{PNA} = \frac{Y_{con} + d_b}{1 + \left(\dfrac{1{,}700\,F_y}{E}\right)} \quad \text{(G3-2)}$$

where

Y_{con} = distance from the top of the steel beam to the top of the concrete, in. (mm)

d_b = depth of the steel beam, in. (mm)

F_y = specified minimum yield stress of the steel beam, ksi (MPa)

E = elastic modulus of the steel beam, ksi (MPa)

5b. Columns

Steel, composite, and reinforced concrete columns shall meet the requirements of Section D1.4a.

5c. Beam Flanges

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 not permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to the required interstory drift angle.

5d. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall meet the requirements of Section D1.3.

User Note: The plastic hinge zones at the ends of C-SMF beams should be treated as *protected zones*. In general, the protected zone will extend from the face of the composite column to one-half of the beam depth beyond the plastic hinge point.

6. Connections

Connections shall be fully-restrained (FR) and shall meet the requirements of Section D2 and this section.

User Note: All subsections of Section D2 are relevant for C-SMF.

6a. Demand Critical Welds

Complete-joint-penetration groove welds of beam flanges, diaphragm plates, shear plates, and beam webs to columns shall be *demand critical* welds as described in Section A3.4b.

6b. Beam-to-Composite Column Connection Requirements

Beam-to-composite column connections used in the *seismic load force resisting system* (SLRS) shall satisfy the following two requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 rad.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an *interstory drift angle* of 0.04 rad, where M_p is calculated as in Section G2.6b.

The two requirements above shall be satisfied as follows:

(1) When beams are interrupted at the connection, the connections shall be qualified using test results obtained in accordance with Section J2. Results of at least two cyclic connection tests shall be provided, and shall be based on one of the following:

(i) Tests reported in research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in J2.

(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 by J2.

(2) When beams are uninterrupted or continuous through the composite or reinforced concrete column, beam flange welded joints are not used, and the connection is not otherwise susceptible to premature fracture, the performance requirements of Section G3.6b shall be demonstrated by qualifying test results in accordance with (1), or other substantiating data.

Connections that accommodate the required *interstory drift angle* within the connection elements and provide the *measured flexural resistance* and shear strengths specified in Section G3.6c are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

6c. Required Shear Strength

The *required shear strength* of the connection, V_u, shall be determined using the following quantity for the earthquake load effect E.

$$E = 2[1.1M_{p,exp}]/L_h \tag{G3-2}$$

where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-encased, or composite beams. For concrete-encased or composite beams, $M_{p,exp}$ shall be calculated according to Section G2.6c, and L_h shall be the distance between plastic hinge locations, in. (mm).

6d. Connection Diaphragm Plates

The continuity plates or diaphragms used for infilled column moment connections shall meet the requirements of Section G2.6d.

6e. Column Splices

Composite column splices shall satisfy the requirements of Section G2.6e.

G4. COMPOSITE PARTIALLY RESTRAINED MOMENT FRAMES (C-PRMF)

1. Scope

Composite partially restrained moment frames (C-PRMF) shall be designed in conformance with this section. This Section is applicable to moment frames that consist of structural steel columns and *composite* *beams* that are connected with partially-restrained (PR) moment connections that meet the requirements in *Specification* Section B3.6b(b).

2. Basis of Design

C-PRMF shall be designed so that yielding occurs in the ductile components of the composite PR beam-to-column moment connections. Limited yielding is permitted at other locations, such as column base connections.

3. Analysis

Analysis shall meet the requirements of Chapter C as modified in this section.

Connection flexibility and composite beam action shall be accounted for in determining the dynamic characteristics, strength and drift of C-PRMF.

For purposes of analysis, the stiffness of beams shall be determined with an effective moment of inertia of the composite section.

4. System Requirements

There are no additional requirements beyond the *Specification*.

5. Members

5a. Columns

Steel columns shall meet the requirements of Section D1.4a and shall meet the requirements of Sections D1.1 for *highly ductile members*.

5b. Beams

Composite beams shall be unencased, fully composite and shall meet the requirements of *Specification* Chapter I. A solid slab shall be provided for a distance of 12 in. around the column.

6. Connections

Connections shall be partially-restrained (PR) and shall meet the requirements of Section D2 and this section.

User Note: All subsections of Section D2 are relevant for C-PRMF.

6a. Required Strength

The *required strength* of the beam-to-column PR moment connections shall be determined considering 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% of M_p, where M_p is the nominal plastic flexural strength of the connected structural steel beam ignoring composite action.

6b. Connection Rotational Capacity

Beam-to-composite column connections used in the SFRS shall satisfy the following requirements:

(1) The connection shall be capable of sustaining an interstory drift angle of at least 0.02 rad.

(2) The *measured flexural resistance* of the connection determined at the column face shall increase monotonically to a value of at least $0.5M_p$ of the connected beam at an *interstory drift angle* of 0.02 rad, where M_p is defined as the nominal flexural strength of the composite beam and shall meet the requirements of *Specification* Chapter I.

The connections shall be qualified for the above requirements using test results obtained in accordance with Section J2. Results of at least two cyclic connection tests shall be provided, and shall be based on one of the following:

(a) Tests reported in research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section J2.

(b) 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 by Section J2.

6c. Connection Strength

Nominal connection strength, $M_{n,PR}$, shall be taken as the bending strength at 0.02 rad during the first excursion at which that rotation is exceeded.

6d. Conformance Demonstration

The ability of the connection to reach the *rotational capacity* specified above shall be substantiated by cyclic testing.

6e. Column Splices

Composite column splices shall satisfy the requirements of Section G2.6e.

CHAPTER H

COMPOSITE BRACED FRAME AND SHEAR WALL SYSTEMS

H1. COMPOSITE ORDINARY BRACED FRAMES (C-OBF)

1. Scope

Composite ordinary braced frames (C-OBF) shall be designed in conformance with this section. Columns shall be structural steel, encased composite, filled composite or reinforced concrete members. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This Section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities are permitted if they are accounted for in the design.

2. Basis of Design

C-OBF 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*. C-OBF shall meet the requirements of Section F1, except as modified in this Section.

User Note: Composite ordinary braced frames, comparable to other steel braced frames designed per the *Specification* using $R = 3$, are only permitted in Seismic Design Categories C or below in SEI/ASCE 7. This is in contrast to steel ordinary braced frames, which are permitted in higher Seismic Design Categories. The design requirements are commensurate with providing minimal ductility in the members and connections.

3. Analysis

There are no additional requirements beyond those in the *Specification* and the *applicable building code*.

4. System Requirements

There are no additional requirements beyond the *Specification* and the *applicable building code*.

5. Members

5a. Detailing Requirements

There are no additional requirements beyond the *Specification* and the *applicable building code.*

5b. Columns

There are no additional requirements beyond the *Specification* and the *applicable building code.* Reinforced concrete columns shall meet the requirements of ACI 318, excluding Chapter 21.

5c. Braces

There are no additional requirements beyond the *Specification* for structural steel and filled composite braces.

6. Connections

Connections shall meet the requirements of Section D2.7 and the *Specification* and the *applicable building code.*

H2. COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-SCBF)

1. Scope

Composite special concentrically braced frames (C-SCBF) shall be designed in conformance with this section. Columns shall be encased or filled composite. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This Section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities are permitted if they are accounted for in the design.

2. Basis of Design

C-SCBF shall be designed to provide inelastic drift capacity through cyclic buckling and tension yielding of braces when subjected to the forces resulting from the motions of the *design earthquake*. SCBF shall be designed so that inelastic deformations under the design earthquake will occur primarily as brace buckling and yielding in tension.

3. Analysis

The analysis requirements for C-SCBF shall satisfy the analysis requirements of Section F2.3.

4. System Requirements

The system requirements for C-SCBF shall satisfy the system requirements of Section F2.4.

5. Members

5a. Detailing Requirements

Steel or composite columns or braces shall meet the requirements of Section D1.1, D1.4b and D1.4c for *highly ductile members.* Steel or composite beams shall meet the requirements of Section D1.1 for *moderately ductile members*.

User Note: In order to satisfy the compactness requirement of Section F2.5a the actual width-to-thickness ratio of square and rectangular filled composite braces may be multiplied by a factor, $[(0.264 + 0.0082Kl/r)]$, for Kl/r between 35 and 90, KL/r being the effective slenderness ratio of the brace.

5b. Columns

Composite columns shall meet the requirements of Section D1.4a.

5c. Braces

Structural steel and filled composite braces shall meet the requirements for SCBF of Section F2.5b. The radius of gyration in Section F2.5b shall be taken as that of the steel section alone.

6. Connections

Design of connections in C-SCBF shall be based on Section D2 and the provisions of this section.

6a. Required Strength of Brace Connections

The required strength of brace connections shall meet the requirements of Section F2.6a, except that design of C-SCBF is limited to the load and resistance factor design (LRFD) method.

6b. Beam-to-Column Connections

Beam-to-column connections shall conform to one of the following:

(1) Beam-to-column connections shall meet the requirements for FR moment connections as specified in Sections D2 and G2.6; or

(2) Beam-to-column connections shall have sufficient rotation capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height.

6c. Column Splices

Column splices in C-SCBF shall be designed following the requirements of Section G2.6e.

H3. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

1. Scope

Composite eccentrically braced frames (C-EBF) shall be designed in conformance with this section. Columns shall be encased composite or filled composite. Beams shall be structural steel. Braces shall be structural steel. This Section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace.

2. Basis of Design

C-EBF shall be designed so that inelastic deformations under the *design earthquake* will occur primarily as shear yielding in the *links*. 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. The *available strength* of members shall meet the requirements in the *Specification*, except as modified in this Section. C-EBF shall meet the requirements of Section F3.2, except as modified in this section.

3. Analysis

The analysis requirements for C-EBF shall satisfy the analysis requirements of Section F3.3.

4. System Requirements

The system requirements for C-EBF shall satisfy the system requirements of Section F3.4.

5. Members

The member requirements of C-EBF shall satisfy the member requirements of Section F3.5 except as noted below.

5a. Links

Links shall be unencased structural steel and shall meet the requirement for *eccentrically braced frame (EBF) links* in Section F3.5a. Beams containing the link are permitted to act compositely with the floor slab using steel anchors along all or any portion of the beam if the composite action is considered when determining the nominal strength of the link.

5b. Columns

Composite columns shall meet the requirements of Section D1.1 for *highly ductile members*.

Composite columns shall meet the requirements of Section D1.4.

6. Connections

The connection requirements of C-EBF shall satisfy the connection requirements of Section F3.6 except as noted below.

6a. Beam-to-Column Connections

Beam-to-column connections shall conform to one of the following:

(1) Beam-to-column connections shall meet the requirements for fully-restrained (FR) ordinary moment connections as specified in Sections D2 and G1.6; or

(2) Beam-to-column connections shall have sufficient rotation capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height.

H4. COMPOSITE ORDINARY REINFORCED CONCRETE SHEAR WALLS WITH STEEL ELEMENTS (C-ORCW)

1. Scope

Composite ordinary reinforced concrete shear walls with steel elements (C-ORCW) shall be designed in conformance with this section. This Section is applicable when reinforced concrete walls are composite with structural steel elements, either as infill panels, such as reinforced concrete walls in structural steel frames where unencased or concrete-encased structural steel sections act as *boundary members* for the walls, or as structural steel or encased composite *coupling beams* that connect two or more adjacent reinforced concrete walls.

2. Basis of Design

Reinforced concrete walls shall meet the requirements of ACI 318 excluding Chapter 21. The design is based on limited inelastic action consistent with ACI 318 excluding Chapter 21.

3. Analysis

Analysis shall meet the requirements of Chapter C as modified in this section.

(a) Uncracked effective stiffness values for elastic analysis shall be assigned according to ACI 318 Chapter 10 for wall piers and composite coupling beams.

(b) When *concrete-encased shapes* function as boundary members, the analysis shall be based upon a transformed concrete section using elastic material properties.

(c) The flexibility of the connection between coupling beams and wall piers and the effect of shear distortions of the coupling beam and walls shall be taken into account.

4. System Requirements

In *coupled walls*, coupling beams are permitted to yield over the height of the structure. The coupling beam–wall connection shall develop the expected flexural and shear strengths of the coupling beam.

In *coupled walls*, it is permitted to redistribute coupling beam forces vertically to adjacent floors. The shear in any individual coupling beam should not be reduced by more than 20% of the elastically determined value. The sum of the coupling beam shear resistance over the height of the building shall be greater than or equal to the sum of the elastically determined values.

5. Members

5a. Boundary Members

Boundary members shall meet the requirements of this Section:

(1) 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.

(2) When the concrete-encased structural steel boundary member qualifies as a *composite column* as defined in *Specification* Chapter I, it shall be designed as a composite column to meet the requirements of Chapter I of the *Specification*.

(3) Headed studs or welded reinforcement anchors shall be provided to transfer required shear strengths between the structural steel boundary members and reinforced concrete walls. Headed studs, if used, shall meet the requirements of *Specification* Chapter I. Welded reinforcement anchors, if used, shall meet the requirements of AWS D1.4.

5b. Steel Coupling Beams

(1) Structural Steel Coupling Beams

Structural steel coupling beams that are used between adjacent reinforced concrete walls shall meet the requirements of the *Specification* and this Section. The requirements apply to wide flange steel coupling beams.

(a) Steel coupling beams shall comply with the requirements of Section D1.1 for *moderately ductile members*.

(b) Coupling beams shall have an embedment length that is sufficient to develop the nominal shear strength, V_n, computed from Equation H4-1.

$$V_n = \frac{2R_yM_p}{g} \le R_yV_p \qquad \text{(H4-1)}$$

where

V_n= nominal shear strength, kips (kN)

$M_p = F_yZ$, kip-in. (N-mm)

V_p= $0.6F_yA_{tw}$, kips (kN)

g = coupling beam clear span, in. (mm)

(c) The embedment length shall be computed from Equation H4-2 and H4-2M. The embedment length shall be considered to begin inside the first layer of confining reinforcement in the wall boundary member.

$$R_yV_n = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left[\frac{0.58-0.22\beta_1}{0.88+\frac{g}{2L_e}}\right] \qquad \text{(H4-2)}$$

$$R_yV_n = 0.004\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left[\frac{0.58-0.22\beta_1}{0.88+\frac{g}{2L_e}}\right] \quad \text{(S.I.)} \qquad \text{(H4-2M)}$$

where

b_w = thickness of wall pier, in. (mm)

b_f = beam flange width, in. (mm)

f'_c = concrete compressive strength, ksi (MPa)

L_e = embedment length, in. (mm)

β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, as defined in ACI 318.

(d) Vertical wall reinforcement with *nominal 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.

(2) Composite Coupling Beams

Encased composite sections serving as *coupling beams* shall meet the requirements of Section H4.5b(1) as modified in this Section:

(a) Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the nominal shear strength, $V_{n,comp}$, computed from Equation H4-3.

$$V_{n,comp} = \frac{2M_{p,exp}}{g} \leq V_{comp} \qquad \text{(H4-3)}$$

where

$M_{p,exp}$ = expected flexural strength of composite coupling beam. For concrete-encased or composite beams, $M_{p,exp}$ shall be calculated using the plastic stress distribution or the strain compatibility method. Appropriate R_y factors shall be used for different elements of the cross-section while establishing section force equilibrium and calculating the flexural strength.

V_{comp} = nominal shear strength of the encased composite coupling beam as computed by Equation H4-4 and H4-4M, ksi (kN)

$$V_{comp} = R_y V_n + \left(2\sqrt{f_c'}\, b_{wc} d_c + \frac{A_s f_{yt} d_c}{s} \right) \qquad \text{(H4-4)}$$

$$V_{comp} = R_y V_n + \left(0.166\sqrt{f_c'}\, b_{wc} d_c + \frac{A_s f_{yt} d_c}{s} \right) \quad \text{(S.I.)} \qquad \text{(H4-4M)}$$

where

A_s = area of transverse reinforcement, in.^{2} (mm^2)

b_{wc} = width of concrete encasement, in. (mm)

d_c = effective depth of concrete encasement

f_{yt} = yield strength of transverse reinforcement, ksi (MPa)

s = spacing of transverse reinforcement, in. (mm)

(b) The required embedment length shall be computed from Equation H4-2 and H4-2M by using $V_{n,comp}$ instead of V_n.

6. Connections

There are no additional requirements beyond Section H4.5.

H5. COMPOSITE SPECIAL REINFORCED CONCRETE SHEAR WALLS WITH STEEL ELEMENTS

1. Scope

Composite special reinforced concrete shear walls composite with steel elements (C-SRCW) shall be designed in conformance with this section.

2. Basis of Design

C-SRCW systems shall meet the requirements of Section H4 and the shear wall requirements of ACI 318 including Chapter 21, except as modified in this Section. The design is based on inelastic actions consistent with Chapter 21 of ACI 318.

3. Analysis

Analysis requirements of Section H4.3 shall be met with the following exceptions.

(a) Cracked effective stiffness values for elastic analysis shall be assigned according to ACI 318 Chapter 10 practice for wall piers and composite coupling beams.

(b) Effects of shear distortion of the steel coupling beam shall be taken into account.

4. System Requirements

System requirements stated in Section H4.4 shall be satisfied with the following exceptions:

(a) In *coupled walls*, coupling beams shall yield over the height of the structure followed by yielding at the base of the wall piers.

(b) In *coupled walls*, the total required compressive axial strength in a wall pier, computed as the sum of the required strengths attributed to the walls from the gravity load components of the lateral load combination plus the sum of the expected beam shear strengths increased by a factor of 1.1 to reflect the effects of strain hardening ($1.1R_yV_n$) of all the coupling beams framing into the walls, shall not exceed the axial design strength of the wall at the balanced condition, P_b.

5. Members

5a. Ductile Elements

Coupling beams are protected zones, and shall satisfy the requirements of Section D1.3. Welding on steel coupling beams is permitted for attachment of stiffeners, as required in Section F3.5a(4).

5b. Boundary Members

Unencased structural steel columns shall meet the requirements of Section H4.5b(1).

In addition to the requirements of Sections H4.3(b) and H4.5a(2), the requirements in this Section shall apply to walls with reinforced-concrete-encased structural steel *boundary members*. Concrete-encased structural steel boundary members that qualify as *composite columns* in *Specification* Chapter I shall meet the *highly ductile member* requirements of Section D1.4b(2). Otherwise, such members shall be designed as composite compression members to meet the requirements of ACI 318 Section 10.13 including the special seismic requirements for boundary members in ACI 318 Section 21.9.6. 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 studs or welded reinforcing bar anchors shall be provided as specified in Section H4.5a(3).

5c. Steel Coupling Beams

In addition to the requirements of Section H4.5b, structural steel *coupling beams* shall meet the requirements of Section F3.5a. When required in Section F3.5a(3), the coupling rotation shall be assumed as 0.08 rad 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 of Section F3.5a(4).

Steel coupling beams shall comply with the requirements of Section D1.1 for highly ductile members.

The expected shear strength for which the embedment length is calculated in Equation H4-1 shall be increased by a factor of 1.1 to reflect the effects of strain hardening ($1.1R_yV_n$).

Vertical wall reinforcement as specified in Section H4.5b(1)(d) shall be confined by transverse reinforcement that meets the requirements for *boundary members* of ACI 318 Section 21.9.6.

Embedded steel members shall be provided with two regions of vertical transfer reinforcement attached to both the top and bottom flanges of the embedded member. The first region shall be located to coincide with the location of longitudinal wall reinforcing bars closest to the face of the wall. The second shall be placed a distance no less than *d/2* from the termination of the embedment length. All transfer reinforcement bars shall be fully developed where they engage the coupling beam flanges. It is permitted to use straight, hooked or mechanical anchorage to provide development. It is permitted to use mechanical couplers welded to the flanges to attach the vertical transfer bars. The area of vertical transfer reinforcement required is computed by Equation H5-1:

$$A_{tb} \geq 0.03 f_c' L_e b_f / f_y \qquad \text{(H5-1)}$$

where

A_{tb} = area of transfer reinforcement required in each of the first and second regions attached to each of the top and bottom flanges, in.^{2} (mm^2)
b_f = beam flange width, in. (mm)
f'_c = concrete compressive strength, ksi (MPa)
f_y = yield strength of transfer reinforcement, ksi (MPa)
L_e = embedment length, in. (mm)

The area of vertical transfer reinforcement shall not exceed that computed by Equation H5-2:

$$\sum A_{tb} < 0.08 L_e b_w - A_s \qquad \text{(H5-2)}$$

where

ΣA_{tb} = total area of transfer reinforcement provided both the first and second regions attached to either the top or bottom flange, in.^{2} (mm^2)
A_s = area of longitudinal wall reinforcement provided over the embedment length L_e, in.^{2} (mm^2)
b_w = wall width, in. (mm)

5d. Composite Coupling Beams

Encased composite sections serving as *coupling beams* shall meet the requirements of Section H5.5c except the requirements of Section F3.5a(3) need not be met, and use Equation H5-3 instead of Equation H4-4. For all composite encased coupling beams,

$$V_e = 1.1R_yV_n + 1.56\left(2\sqrt{f_c'}b_{wC}d_C + \frac{A_s f_{yr} d_C}{s}\right) \quad \text{(H5-3)}$$

$$V_e = 1.1R_yV_n + 1.56\left(0.166\sqrt{f_c'}b_{wC}d_C + \frac{A_s f_{yr} d_C}{s}\right) \quad \text{(S.I.)} \quad \text{(H5-3M)}$$

6. Connections

Groove welds at column splices shall be demand critical welds as described in Section A3.4b.

H6. COMPOSITE STEEL PLATE SHEAR WALLS

1. Scope

Composite steel plate shear walls (C-SPW) shall be designed in conformance with this section. *Composite steel plate shear walls* consist of steel plates with reinforced concrete encasement on one or both sides of the plate, or steel plates on both sides of reinforced concrete infill, and structural steel or composite *boundary members*.

2. Basis of Design

C-SPW are expected to withstand significant inelastic deformations in the webs. The horizontal boundary elements (HBE) and vertical boundary elements (VBE) adjacent to the composite webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded steel webs along with the reinforced concrete webs after the steel web has fully yielded, except that plastic hinging at the ends of HBE is permitted.

3. Analysis

3a. Webs

Steel webs shall be designed to resist the seismic load *E* determined from the analysis required by the applicable building code. The analysis shall account for openings in the web.

3b. Other Members and Connections

Columns, beams, and connections in C-SPW shall be designed to resist seismic forces determined from an analysis that includes the expected strength of the steel webs in shear, $0.6R_yF_yA_{sp}$, and any reinforced concrete portions of the wall active at the design story drift. The vertical boundary elements (VBE) are permitted to yield at the base.

4. System Requirements

4a. Steel Plate Thickness

Steel plates with thickness less than 3/8 in. (9.5 mm) are not permitted.

4b. Stiffness of Vertical Boundary Elements

The VBE shall meet the requirements of Section F5.4a.

4c. HBE-to-VBE Connection Moment Ratio

The beam-column moment ratio shall meet the requirements of Section F5.4b.

4d. Bracing

The bracing shall meet the requirements of Section F5.4c.

4e. Openings in Webs

Boundary members shall be provided around openings in shear wall webs as required by analysis.

4f. Detailing Requirements

Steel and composite HBE and VBE shall meet the requirements of Section D1.1 for *highly ductile members*.

5. Members

5a. Webs

The *design shear strength*, ϕV_{ns}, for the limit state of shear yielding with a composite plate conforming to Section H6.5b shall be taken as:

$$V_n = 0.6A_{sp}F_y \quad \text{(H6-1)}$$

$$\phi = 0.90 \text{ (LRFD)}$$

where

V_n = 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 stress of the plate, ksi (MPa)

The *design shear strength* of C-SPW with a plate that does not meet the stiffening requirements in Section H6.5b shall be based upon the strength of the plate as given in Section F5.5 and meet the requirements of

Specification Sections G2 and G3.

5b. Concrete Stiffening Elements

The steel plate shall be adequately stiffened by encasement or attachment to a reinforced concrete panel. Conformance to this requirement shall be demonstrated with an elastic plate buckling analysis showing that the composite wall can resist a nominal shear force equal to V_{ns}.

The concrete thickness shall be a minimum of 4 in. (100 mm) on each side when concrete is provided on both sides of the steel plate and 8 in. (200 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 or exceed the 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. (450 mm).

5c. Boundary Members

Structural steel and composite *boundary members* shall be designed to resist the *expected shear strength* of steel plate and any reinforced concrete portions of the wall active at the design story drift. Composite and reinforced concrete boundary members shall also meet the requirements of Section H5.5b. Steel boundary members shall also meet the requirements of Section F5.

5d. Detailing Requirements

HBE and VBE shall meet the requirements of Section D1.1 for *highly ductile members*.

6. Connections

6a. Demand Critical Welds

Welds that connect the shear wall webs to the boundary elements and groove welds at column splices shall be *demand critical* welds as described in Section A3.4b.

6b. Connections of Steel Plate to Boundary Elements

The steel plate shall be continuously welded or bolted on all edges to the structural steel framing and/or steel *boundary members*, or the steel component of the composite boundary members. Welds and/or slip-critical high-strength bolts are required to develop the nominal shear strength of the plate shall be provided.

6c. HBE-to-VBE Connections

HBE-to-VBE connections shall meet the requirements of Section F5.

6d. Connections of Steel Plate to Reinforced Concrete Panel

The shear connectors between the steel plate and the reinforced concrete panel shall be designed to prevent its overall buckling. Shear connectors shall be designed to satisfy the following conditions:

(1) Tension in the connector

The shear connector shall be designed to resist the tension force resulting from inelastic local buckling of the steel plate.

(2) Shear in the connector

The shear connectors collectively shall be designed to transfer expected strength in shear of steel plate or reinforced concrete panel, whichever is smaller.

CHAPTER I

FABRICATION, ERECTION, QUALITY CONTROL AND QUALITY ASSURANCE

This chapter addresses requirements for fabrication, erection, quality control and quality assurance. All requirements of *Specification* Section M shall apply, unless specifically modified by these *Provisions*.

The chapter is organized as follows:

I1. Shop and Erection Drawings
I2. Fabrication and Erection
I3. Quality Control and Quality Assurance

I1. SHOP AND ERECTION DRAWINGS

1. Shop Drawings for Steel Construction

Shop drawings and erection drawings shall indicate the work to be performed, and include items required by the *Specification*, the AISC *Code of Standard Practice for Steel Buildings and Bridges*, the *applicable building code*, the requirements of Sections A4.1 and A4.2, and the following, as applicable:

(1) Locations of pretensioned bolts
(2) Locations of Class A, or better, faying surfaces
(3) Gusset plates drawn to scale when they are designed to accommodate inelastic rotation
(4) Weld access hole dimensions, surface profile and finish requirements
(5) Locations where weld backing is required to be removed
(6) Locations where fillet welds are required where backing is permitted to remain
(7) Locations where fillet welds are required to reinforce groove welds, or where fillet welds are used to improve connection geometry
(8) Locations where weld tabs are required to be removed
(9) Nondestructive testing (NDT) where performed by the fabricator

2. Erection Drawings for Steel Construction

Erection drawings shall indicate the work to be performed, and include items required by the *Specification*, the AISC *Code of Standard Practice* *for Steel Buildings and Bridges*, the *applicable building code*, the requirements of Sections A4.1 and A4.2, and the following, as applicable:

(1) Locations of pretensioned bolts
(2) Locations where weld backing is to be removed.
(3) Locations where fillet welds are required where backing is permitted to remain
(4) Locations where fillet welds are required to reinforce groove welds, or where fillet welds are used to improve connection geometry.
(5) Locations where weld tabs are to be removed.
(6) Those joints or groups of joints in which a specific assembly order, welding sequence, welding technique or other special precautions are required.

3. Shop and Erection Drawings for Composite Construction

Shop drawings and erection drawings for the steel components of composite steel-concrete construction shall meet the requirements of Sections I1.1 and I1.2.

For reinforced concrete and composite steel-concrete construction, the provisions of ACI 315-99 *Details and Detailing of Concrete Reinforcement*, ACI 315-R-94 *Manual of Engineering and Placing Drawings for Reinforced Concrete Structures*, and ACI SP-66 *ACI Detailing Manual*, including modifications required by Chapter 21 of ACI 318-05, and ACI 352 *Monolithic Joints in Concrete Structures* shall be satisfied. The shop drawings and erection drawings shall meet the requirements of Section A4.3.

I2. FABRICATION AND ERECTION

1. Protected Zone

Where a protected zone is designated by these *Provisions* or ANSI/AISC 358, it shall comply with the following:

(i) Within the protected zone, discontinuities created by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging and thermal cutting, shall be repaired as required by the *engineer of record*.

(ii) Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on beam flanges within the protected zone. Arc spot welds as required to secure decking shall be permitted.

(iii) 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 protected zone.

Exception: Other attachments are permitted where designated or approved by the engineer of record. See Section D1.3.

User Note: AWS D1.8 clause 6.15 contains requirements for weld removal and the repair of gouges and notches in the protected zone.

2. Bolted Joints

Bolted joints shall satisfy the requirements of Section D2.2.

3. Welded Joints

Welding shall be performed in accordance with a welding procedure specification (WPS) as required in *Structural Welding Code–Steel* (AWS D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M, and approved by the engineer of record. The WPS variables shall be within the parameters established by the filler metal manufacturer.

In addition to the above requirements, *demand critical welds* shall be made in accordance with AWS D1.8 clauses 6.1.2, 6.1.3, 6.2.1 and 6.4.

Welds shall be made using materials as prescribed in Section A3.4.

Welding details, welding personnel qualification, fabrication and erection shall satisfy the requirements of *Structural Welding Code – Seismic Supplement* (AWS D1.8), hereafter referred to as AWS D1.8, except as follows:

(1) Weld tab removal shall meet the requirements of AWS D1.8, except at the outboard ends of weld tabs for continuity plates, where ¼ in. (6 mm) of weld tab material and weld metal is permitted to remain. See Section I2.5.

(2) AWS D1.8 clauses relating to fabrication shall apply equally to shop fabrication welding and to field erection welding.

User Note: AWS D1.8 was specifically written to provide additional requirements for the welding of seismic force resisting systems, and has been coordinated wherever possible with these *Provisions*. AWS D1.8 requirements related to fabrication and erection are organized as follows, including normative (mandatory) annexes:
1. General Requirements
2. Reference Documents
3. Definitions

4. Welded Connection Details
5. Welder Qualification
6. Fabrication

Annex A. WPS Heat Input Envelope Testing of Filler Metals for Demand Critical Welds

Annex B. Intermix CVN Testing of Filler Metal Combinations (where one of the filler metals is FCAW-S)

Annex C. Supplemental Welder Qualification for Restricted Access Welding

Annex D. Supplemental Testing for Extended Exposure Limits for FCAW Filler Metals

AWS D1.8 requires the complete removal of all weld tab material, leaving only base metal and weld metal at the edge of the joint. This is to remove any weld discontinuities at the weld ends, as well as facilitate magnetic particle testing (MT) of this area. At continuity plates, these *Provisions* permit a limited amount of weld tab material to remain because of the reduced strains at continuity plates, and any remaining weld discontinuities in this weld end region would likely be of little significance. Also, weld tab removal sites at continuity plates are not subjected to MT.

AWS D1.8 Clause 6 is entitled “Fabrication, but the intent of AWS is that all provisions of AWS D1.8 apply equally to fabrication and erection activities as described in the *Specification* and in these *Provisions*.

4. Continuity Plates and Stiffeners

Corners of continuity plates and stiffeners placed in the webs of rolled shapes shall be detailed in accordance with AWS D1.8 Clause 4.1.

5. Weld Tabs

Weld tabs shall be in accordance with AWS D1.8 Clause 6.10.

Exception: At the outboard ends of continuity plate to column welds, weld tabs and weld metal need not be removed closer than ¼ in. (6 mm) from the continuity plate edge.

I3. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control (QC) and quality assurance (QA) shall be provided as specified in this Section.

When required by the applicable building code or contract documents, the engineer of record shall prepare a quality assurance plan (QAP). The quality assurance plan shall identify the specific quality control and quality assurance tasks to be performed on the project respectively by the contractor and the QA agency. The contractor and quality assurance agency shall perform those tasks as identified in the QAP. The QAP shall include the requirements of this section.

User Note: The quality assurance plan in Section I3 is considered adequate and effective for most seismic force resisting systems and should be used without modification. The quality assurance plan is intended to ensure that the seismic force resisting system is significantly free of defects that would greatly reduce the ductility of the system. There may be cases (for example, nonredundant major transfer members, or where work is performed in a location that is difficult to access) where supplemental testing might be advisable. Additionally, where the contractor's quality control program has demonstrated the capability to perform some tasks this plan has assigned to quality assurance, modification of the plan could be considered.

1. Inspection and Nondestructive Testing Personnel

Visual welding inspection and nondestructive testing (NDT) shall be conducted in accordance with a written practice by personnel qualified in accordance with AWS D1.8 Clause 7.2.

Bolting inspection shall be conducted in accordance with a written practice by qualified personnel.

User Note: The recommendations of the ICC Model Program for Special Inspection should be considered a minimum requirement to establish the qualifications of a bolting inspector.

2. Contractor Documents

2a. Documents to be Submitted for Steel Construction

The following documents shall be submitted for review by the *engineer of record* or designee, prior to fabrication or erection, as applicable:

(1) Shop drawings
(2) Erection drawings
(3) Welding procedure specifications (WPS)
(4) Copies of the manufacturer's typical certificate of conformance for all electrodes, fluxes and shielding gasses to be used.
(5) For demand critical welds, applicable manufacturer's certifications that the filler metal meets the supplemental notch toughness requirements, as applicable. Should the filler metal manufacturer not supply such supplemental certifications, the contractor shall have the necessary testing performed and provide the applicable test reports.
(6) Manufacturer's product data sheets or catalog data for SMAW,

FCAW and GMAW composite (cored) filler metals to be used.

(7) Bolt installation procedures

(8) Specific assembly order, welding sequence, welding technique or other special precautions for joints or groups of joints where such items are designated to be submitted to the engineer of record

2b. Documents to be Available for Review for Steel Construction

The following documents shall be available for review by the engineer of record or designee prior to fabrication or erection, as applicable, unless specified to be submitted:

(1) Material test reports for structural steel, bolts, shear connectors, and welding materials

(2) Inspection procedures

(3) Nonconformance procedure

(4) Material control procedure

(5) Welder performance qualification records (WPQRs), including any supplemental testing requirements

(6) QC Inspector qualifications

The contractor shall retain the document(s) for at least one year after substantial completion of construction.

2c. Documents to be Submitted for Composite Construction

The following documents shall be submitted for review by the engineer of record or designee, prior to concrete production or placement, as applicable:

(1) Concrete mix design and test reports for the mix design

(2) Reinforcing steel shop drawings

(3) Concrete placement sequences, techniques and restriction

2d. Documents to be Available for Review for Composite Construction

The following documents shall be available for review by the engineer of record or designee prior to fabrication or erection, as applicable, unless specified to be submitted:

(1) Material test reports for reinforcing steel

(2) Inspection procedures

(3) Nonconformance procedure

(4) Material control procedure

(5) Welder performance qualification records (WPQRs) as required by AWS D1.4

(6) QC Inspector qualifications

The contractor shall retain the document(s) for at least one year after substantial completion of construction.

3. Quality Assurance Agency Documents

The agency responsible for quality assurance shall submit the following documents to the authority having jurisdiction, the engineer of record, and the owner or owner's designee:

(1) QA agency's written practices for the monitoring and control of the agency's operations. The written practice shall include:
  (i) The agency's procedures for the selection and administration of inspection personnel, describing the training, experience and examination requirements for qualification and certification of inspection personnel, and
  (ii) The agency's inspection procedures, including general inspection, material controls, and visual welding inspection.

(2) Qualifications of management and QA personnel designated for the project.

(3) Qualification records for Inspectors and NDT technicians designated for the project.

(4) NDT procedures and equipment calibration records for NDT to be performed and equipment to be used for the project.

(5) For composite construction, concrete testing procedures and equipment

(6) Daily or weekly inspection reports.

(7) Nonconformance reports.

4. Inspection Points and Frequencies

Inspection points and frequencies of quality control (QC) and quality assurance (QA) tasks and documentation for the seismic force resisting system (SFRS) shall be as provided in the following tables.

The following entries are used in the tables:

4a. Observe (O)

The inspector shall observe these functions on a random, daily basis. Welding operations need not be delayed pending observations. For shop fabrication, where a task is noted to be performed by both QC and QA, the frequency of QA inspections is permitted to be reduced following demonstrated performance by the fabricator in meeting the requirements of these *Provisions*, as evaluated and approved by the engineer of record and the authority having jurisdiction.

4b. Perform (P)

These inspections shall be performed prior to the final acceptance of the item. Where a task is noted to be performed by both QC and QA, coordination of the inspection function between QC and QA is permitted so that the inspection functions need be performed by only one party. Where QA is to rely upon inspection functions performed by QC, the approval of the engineer of record and the authority having jurisdiction is required.

4c. Document (D)

The inspector shall prepare reports indicating that the work has been performed in accordance with the contract documents. The report need not provide detailed measurements for joint fit-up, WPS settings, completed welds, or other individual items listed in the Tables in Section I3.5a. For shop fabrication, the report shall indicate the piece mark of the piece inspected. For field work, the report shall indicate the reference grid lines and floor or elevation inspected. Work not in compliance with the contract documents and whether the noncompliance has been satisfactorily repaired shall be noted in the inspection report.

5. Welding Inspection

Welding inspection and nondestructive testing shall satisfy the requirements of this section and AWS D1.8.

User note: AWS D1.8 was specifically written to provide additional requirements for the welding of seismic force resisting systems, and has been coordinated when possible with these *Provisions*. AWS D1.8 requirements related to inspection and nondestructive testing are organized as follows, including normative (mandatory) annexes:
1. General Requirements
7. Inspection
Annex E. Supplemental Ultrasonic Technician Testing
Annex F. Supplemental Magnetic Particle Testing Procedures
Annex G. Flaw Sizing by Ultrasonic testing

5a. Visual Welding Inspection

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. All requirements of AWS D1.1/D1.1M for statically loaded structures shall apply to the designated welds, except as specifically modified by AWS D1.8.

Visual welding inspection shall be performed by both quality control and quality assurance personnel. As a minimum, tasks shall be as listed in Tables I3-1, I3-2 and I3-3.

TABLE I3-1
Visual Inspection Tasks Prior to Welding

Visual Inspection Tasks Prior to Welding	QC		QA	
	Task	Doc.	Task	Doc.
Material identification (Type/Grade)	O	-	O	-
Welder identification system	O	-	O	-
Fit-up of Groove Welds (including joint geometry) - Joint preparation - Dimensions (alignment, root opening, root face, bevel) - Cleanliness (condition of steel surfaces) - Tacking (tack weld quality and location) - Backing type and fit (if applicable)	P/O**	-	O	-
Configuration and finish of access holes	O	-	O	-
Fit-up of Fillet Welds - Dimensions (alignment, gaps at root) - Cleanliness (condition of steel surfaces) - Tacking (tack weld quality and location)	P/O**	-	O	-

** Following performance of this inspection task for ten welds to be made by a given welder, with the welder demonstrating adequate understanding of requirements and possession of skills and tools to verify these items, the Perform designation of this task shall be reduced to Observe, and the welder shall perform this task. Should the inspector determine that the welder has discontinued adequate performance of this task, the task shall be returned to Perform until such time as the Inspector has re-established adequate assurance that the welder will perform the inspection tasks listed.

TABLE I3-2
Visual Inspection Tasks during Welding

Visual Inspection Tasks During Welding	QC		QA	
	Task	Doc.	Task	Doc.
WPS followed - Settings on welding equipment - Travel speed - Selected welding materials - Shielding gas type/flow rate - Preheat applied - Interpass temperature maintained (min/max.) - Proper position (F, V, H, OH) - Intermix of filler metals avoided unless approved	O	-	O	-
Use of qualified welders	O	-	O	-
Control and handling of welding consumables - Packaging - Exposure control	O	-	O	-
Environmental conditions - Wind speed within limits - Precipitation and temperature	O	-	O	-
Welding techniques	O	-	O	-

- Interpass and final cleaning - Each pass within profile limitations - Each pass meets quality requirements				
No welding over cracked tacks	O	-	O	-

TABLE I3-3
Visual Inspection Tasks after Welding

Visual Inspection Tasks After Welding	QC		QA	
	Task	Doc.	Task	Doc.
Welds cleaned	O	-	O	-
Size length, and location of welds	P	-	P	-
Welds meet visual acceptance criteria - Crack prohibition - Weld/base-metal fusion - Crater cross-section - Weld profiles and size - Undercut - Porosity	P	D	P	D
Placement of reinforcement fillets	P	D	P	D
Backing removed, weld tabs removed and finished, <u>and fillet welds added</u> (if required)	P	D	P	D
Repair activities	P	-	P	D

5b. Nondestructive Testing (NDT) of Welds

Nondestructive testing of welds as described in the following shall be performed by quality assurance personnel:

(1) k-Area NDT

Where welding of doubler plates, continuity plates, or stiffeners has been performed in the k-area, the web shall be tested for cracks using magnetic particle testing (MT). The MT inspection area shall include the k-area base metal within 3 in. (75 mm) of the weld. The MT shall be performed no sooner than 48 hours following completion of the welding.

(2) CJP Groove Weld NDT

Ultrasonic testing (UT) shall be performed on 100% of CJP groove welds in materials $\frac{5}{16}$ in. (8 mm) thick or greater. Ultrasonic testing in materials less than $\frac{5}{16}$ in. (8 mm) thick is not required. Weld discontinuities shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2. Magnetic particle testing shall be performed on 25% of all beam-to-column CJP groove welds. The rate of UT and MT is permitted to be reduced in accordance with Section I3.5b(7) and (8), respectively.

Exception: For ordinary moment frames, UT and MT of CJP groove welds are required only for demand critical welds.

(3) Base Metal NDT for Lamellar Tearing and Laminations

After joint completion, base metal thicker than 1½ in. (38 mm) loaded in tension in the through thickness direction in tee and corner joints, where the connected material is greater than ¾ in. (19 mm) and contains CJP groove welds, shall be ultrasonically tested for discontinuities behind and adjacent to the fusion line of such welds. Any base metal discontinuities found within $t/4$ of the steel surface shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2, where t is the thickness of the part subjected to the through-thickness strain.

(4) Beam Cope and Access Hole NDT

At welded splices and connections, thermally cut surfaces of beam copes and access holes shall be tested using magnetic particle testing or penetrant testing, when the flange thickness exceeds 1½ in. (38 mm) for rolled shapes, or when the web thickness exceeds 1½ in. (38 mm) for built-up shapes.

(5) Reduced Beam Section Repair NDT

Magnetic particle testing shall be performed on any weld and adjacent area of the reduced beam section (RBS) cut surface that has been repaired by welding, or on the base metal of the RBS cut surface if a sharp notch has been removed by grinding.

(6) Weld Tab Removal Sites

At the end of welds where weld tabs have been removed, magnetic particle testing shall be performed on the same beam-to-column joints receiving UT as required under Section I3.5b(2). The rate of MT is permitted to be reduced in accordance with Section I3.5b(8). MT of continuity plate weld tabs removal sites is not required.

(7) Reduction of Percentage of Ultrasonic Testing

Except for demand critical welds, the amount of ultrasonic testing is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The nondestructive testing rate for an individual welder or welding operator is permitted to be reduced to 25%, provided the reject rate is demonstrated to be 5% or less of the welds tested for the welder or welding operator.

A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is the number of welds containing rejectable defects divided by the number of welds completed. For evaluating the reject rate of continuous welds over 3 ft (1 m) in length where the effective throat thickness is 1 in. (25 mm) or less, each 12 in. (300 mm) increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 ft (1 m) in length where the effective throat thickness is greater than 1 in. (25 mm), each 6 in. (150 mm) of length or fraction thereof shall be considered one weld.

(8) Reduction of Percentage of Magnetic Particle Testing

The amount of MT on CJP groove welds is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The MT rate for an individual welder or welding operator is permitted to be reduced to 10%, provided the reject rate is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds for a job shall be made for such reduction evaluation. Reject rate is the number of welds containing rejectable defects divided by the number of welds completed. This reduction is prohibited on welds in the k-area, at repair sites, backing removal sites, and access holes.

(9) Documentation

All NDT performed shall be documented. For shop fabrication, the NDT report shall identify the tested weld by piece mark and location in the piece. For field work, the NDT report shall identify the tested weld by location in the structure, piece mark, and location in the piece.

6. Inspection of Bolting

Observation of bolting operations 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. As a minimum, the tasks shall be as listed in Tables I3-4, I3-5 and I3-6.

TABLE I3-4
Inspection Tasks Prior To Bolting

Inspection Tasks Prior To Bolting	QC		QA	
	Task	Doc.	Task	Doc.

Inspection Tasks Prior To Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Proper bolts selected for the joint detail	O	-	O	-
Proper bolting procedure selected for joint detail	O	-	O	-
Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements	O	-	O	-
Pre-installation verification testing observed for fastener assemblies and methods used	P	D	O	D
Proper storage provided for bolts, nuts, washers and other fastener components	O	-	O	-

TABLE I3-5
Inspection Tasks during Bolting

Inspection Tasks During Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Fastener assemblies placed in all holes and washers (if required) are positioned as required	O	-	O	-
Joint brought to the snug tight condition prior to the pretensioning operation	O	-	O	-
Fastener component not turned by the wrench prevented from rotating	O	-	O	-
Bolts are pretensioned progressing systematically from most rigid point toward free edges	O	-	O	-

TABLE I3-6
Inspection Tasks after Bolting

Inspection Tasks After Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Document accepted and rejected connections	P	D	P	D

7. Other Steel Structure Inspections

Where applicable, the following inspection tasks listed in Table I3-7 shall be performed:

TABLE I3-7
Other Inspection Tasks

Other Inspection Tasks	QC		QA	
	Task	Doc	Task	Doc.
RBS requirements, if applicable - contour and finish - dimensional tolerances	P	D	P	D
Protected zone - no holes and unapproved attachments made by	P	D	P	D

contractor				

User Note: The protected zone should be inspected by others following completion of the work of other trades, including those involving curtainwall, mechanical, electrical, plumbing and interior partitions.

8. Inspection of Composite Structures

Where applicable, inspection of structural steel used in composite structures shall comply with the requirements of Chapter I.

Where applicable, inspection of reinforced concrete shall comply with the requirements of ACI 318 and the applicable requirements of Section I3.5a for welded reinforcing steel.

Where applicable to the type of composite construction, the following minimum inspection tasks shall be as listed in Tables I3-8, I3-9 and I3-10.

TABLE I3-8
Inspection of Composite Structures Prior to Concrete Placement

Inspection of Composite Structures Prior to Concrete Placement	QC		QA	
	Task	Doc	Task	Doc.
Material identification of reinforcing steel (Type/Grade)	O	-	O	-
Confirmation of acceptable carbon equivalent for reinforcing steel not complying with requirements of ASTM A706	O	-	O	-
Confirmation of proper reinforcing steel size, spacing and orientation	O	-	O	--
Confirmation that reinforcing steel has not been rebent in the field	O	-	O	-
Confirmation that reinforcing steel has been tied and supported as required	O	-	O	-
Confirmation that required reinforcing steel clearances have been provided	O	-	O	-
Confirmation that composite member has required size	O	-	O	-

TABLE I3-9
Inspection of Composite Structures during Concrete Placement

Inspection of Composite Structures during Concrete Placement	QC		QA	
	Task	Doc	Task	Doc.
Concrete: Material identification (mix design, compressive strength, maximum large aggregate size, maximum slump)	O	D	O	D
Limits on water added at the truck or pump	O	D	O	D
Proper placement techniques to limit segregation	O	-	O	-

TABLE I3-10
Inspection of Composite Structures after Concrete Placement

Inspection of Composite Structures After Concrete Placement	QC		QA	
	Task	Doc	Task	Doc.
Achievement of minimum specified concrete compressive strength at specified age	-	D	-	D

9. Inspection of Piling

Where applicable, the following inspection tasks listed in Table I3-11 shall be performed:

TABLE I3-11
Inspection of Piling

Inspection of Piling	QC		QA	
	Task	Doc	Task	Doc.
Protected zone - no holes and unapproved attachments made by contractor	P	D	P	D

CHAPTER J

QUALIFICATIONS AND PREQUALIFICATION TESTING PROVISIONS

This chapter addresses requirements for qualifications and prequalification testing. This chapter is organized as follows:

J1. PREQUALIFICATION OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

1. Scope

This section contains minimum requirements for prequalification of beam-to-column moment connections in special moment frames (SMF), intermediate moment frames (IMF), and link-to-column connections in eccentrically braced frames (EBF). Prequalified connections are permitted to be used, within the applicable limits of prequalification, without the need for further qualifying cyclic tests. When the limits of prequalification or design requirements for prequalified connections conflict with the requirements of these Provisions, the limits of prequalification and design requirements for prequalified connections shall govern.

2. General Requirements

2a. Basis for Prequalification

Connections shall be prequalified based on test data satisfying Section J1.3, 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 EBF, 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 force* *resisting system (SFRS)* must be identified. These include rupture 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 J1.4 shall be addressed for connection prequalification.

2b. 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.

3. Testing Requirements

Data used to support connection prequalification shall be based on tests conducted in accordance with Section J2. 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 a sufficient number of nonidentical specimens to demonstrate that the connection has the ability and reliability to undergo the required interstory drift angle for SMF and IMF and the required link rotation angle for EBF, where the link is adjacent to columns. The limits on member sizes for prequalification shall not exceed the limits specified in Section J2.3b.

4. 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.

4a. Beam or Link Parameters

(1) Cross-section shape: wide flange, box, or other.

(2) Cross-section fabrication method: rolled shape, welded shape, or other.

(3) Depth.

(4) Weight per foot.

(5) Flange thickness.

(6) Material specification.

(7) Span-to-depth ratio (for SMF or IMF), or link length (for EBF).

(8) Width-thickness ratio of cross-section elements.

(9) Lateral bracing.

(10) Other parameters pertinent to the specific connection under consideration.

4b. Column Parameters

(1) Cross-section shape: wide flange, box, or other.

(2) Cross-section fabrication method: rolled shape, welded shape, or other.

(3) 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.

(4) Depth.

(5) Weight per foot.

(6) Flange thickness.

(7) Material specification.

(8) Width-thickness ratio of cross-section elements.

(9) Lateral bracing.

(10) Other parameters pertinent to the specific connection under consideration.

4c. Beam (or Link) – Column Relations

(1) Panel zone strength.

(2) Doubler plate attachment details.

(3) Column-beam (or link) moment ratio.

4d. Continuity Plates

(1) Identification of conditions under which continuity plates are required.

(2) Thickness, width and depth.

(3) Attachment details.

4e. Welds

(1) Location, extent (including returns), type (CJP, PJP, fillet, etc.) and any reinforcement or contouring required.

(2) Filler metal classification strength and notch toughness.

(3) Details and treatment of weld backing and weld tabs.

(4) Weld access holes: size, geometry and finish.

(5) Welding quality control and quality assurance beyond that described in Chapter I, including NDT method, inspection frequency, acceptance criteria and documentation requirements.

4f. Bolts

(1) Bolt diameter.

(2) Bolt grade: ASTM A325, A490, or other.

(3) Installation requirements: pretensioned, snug-tight, or other.

(4) Hole type: standard, oversize, short-slot, long-slot, or other.

(5) Hole fabrication method: drilling, punching, sub-punching and reaming, or other.

(6) Other parameters pertinent to the specific connection under consideration.

4g. Workmanship

All workmanship parameters that exceed AISC, RCSC and AWS requirements, pertinent to specific connection under consideration, as follows:

(1) Surface roughness of thermal cut or ground edges.

(2) Cutting tolerances.

(3) Presence of holes, fasteners or welds for attachments.

4h. Additional Connection Details

All variables pertinent to the specific connection under consideration, as established by the CPRP.

5. 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.

6. 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 J1.4.

(5) Listing of demand critical welds.

(6) Definition of the region of the connection that comprises the protected zone.

(7) Detailed description of the design procedure for the connection, as required in Section J1.5.

(8) List of references of test reports, research reports and other publications that provided the basis for prequalification.

(9) Summary of quality control and quality assurance procedures.

J2. CYCLIC TESTS FOR QUALIFICATION OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

1. Scope

This section includes requirements for qualifying cyclic tests of beam-to-column moment connections in special and intermediate moment frames and link-to-column connections in *eccentrically braced frames*, when required in these *Provisions*. The purpose of the testing described in this section 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 section provides minimum recommendations for simplified test conditions.

2. 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.

3. 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.

3a. Sources of Inelastic Rotation

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 special and intermediate moment frames, inelastic rotation is 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.

Inelastic rotation shall be developed in the test specimen by inelastic action in the same members and connection elements as anticipated in the prototype (in other words, in the beam or link, in the column panel zone, in the column outside of the panel zone, or in connection elements) within the limits described below. The percentage of the total inelastic rotation in the test specimen that is developed in each member or connection element shall be within 25% of the anticipated percentage of the total inelastic rotation in the prototype that is developed in the corresponding member or connection element.

3b. 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% 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% 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 J2.3a. In addition, the depth of the test column shall be no less than 90% 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*.

User Note: Based upon the above criteria, beam depth and column depths up to and including 11% greater than that tested should be permitted for the prototype. Weight per foot of the beam or link up to and including 33% greater than that tested should be permitted for the prototype.

3c. 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.

3d. 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.

3e. Steel 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 strength shall be determined as specified in Section J2.6a. The use of yield stress values that are reported on certified material test reports in lieu of physical testing is prohibited for purposes of this Section.

(2) The yield strength of the beam flange as tested in accordance with Section J2.6a shall not be more than 15% below R_yF_y for the grade of steel to be used for the corresponding elements of the prototype.

(3) The yield strength of the columns and connection elements shall not be more than 15% 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 A3.2.

User Note: Based upon the above criteria, steel of the specified grade with a F_y of up to and including 1.15 times the R_yF_y for the steel tested should be permitted in the prototype. In production, this limit should be checked using the values stated on the steel manufacturer's material test reports.

3f. Welded Joints

Welds on the test specimen shall satisfy the following requirements:

(1) Welding shall be performed in conformance with Welding Procedure Specifications (WPS) as required in AWS D1.1/D1.1M. The WPS essential variables shall meet the requirements in AWS D1.1/D1.1M and shall be within the parameters established by the filler-metal manufacturer. The tensile strength and Charpy V-notch (CVN) toughness of the welds used in the *test assembly* shall be determined by tests as specified in Section J2.6c, made using the same filler metal classification, manufacturer, brand or trade name, diameter, and average heat input for the WPS used on the *test specimen*. The use of tensile strength and CVN toughness values that are reported on the manufacturer's typical certificate of conformance in lieu of physical testing is prohibited for purposes of this section.

(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 welds on the corresponding prototype. The tensile strength of the deposited weld as tested in accordance

with Section J2.6c shall not exceed the tensile strength classification of the filler metal specified for the prototype by more than 25 ksi (125 MPa).

User Note: Based upon the criteria in (2) above, should the tested tensile strength of the weld metal exceed 25 ksi (125 MPa) above the specified minimum tensile strength, the prototype weld should be made with a filler metal and WPS that will provide a tensile strength no less than 25 ksi below the tensile strength measured in the *material test plate*. When this is the case, the tensile strength of welds resulting from use of the filler metal and the WPS to be used in the prototype should be determined by using an all-weld-metal tension specimen. The test plate is described in AWS D1.8 clause A6 and shown in AWS D1.8 Figure A.1.

(3) The specified minimum CVN toughness of the filler metal used for the test specimen shall not exceed that to be used for the welds on the corresponding prototype. The tested CVN toughness of the weld as tested in accordance with Section J2.6c shall not exceed the minmum CVN toughness specified for the prototype by more than 50%, nor 25 ft-lbs (34 kJ), whichever is greater.

User Note: Based upon the criteria in (3) above, should the tested CVN toughness of the weld metal in the material test specimen exceed the specified CVN toughness for the *test specimen* by 25 ft-lbs (34 kJ) or 50%, whichever is greater, the prototype weld should be made with a filler metal and WPS that will provide a CVN toughness that is no less than 25 ft-lbs (34 kJ) or 33% lower, whichever is lower, below the CVN toughness measured in the *material test plate*. When this is the case, the weld properties resulting from the filler metal and WPS to be used in the prototype should be determined using five CVN test specimens. The test plate is described in AWS D1.8 clause A6 and shown in AWS D1.8 Figure A.1.

(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.

User Note: The filler metal used for production of the prototype is permitted to be of a different classification, manufacturer, brand or trade name, and diameter, provided that Sections J2.3f(3) and J2.3f(4) are satisfied. To qualify alternate filler metals, the tests as presecribed in Section J2.6c should be conducted.

3g. Bolted Joints

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 (for example, ASTM A325, A325M, ASTM A490, A490M, ASTM F1852, ASTM F2280) used in the test specimen shall be the same as that to be used for the prototype, except that heavy hex bolts may be substituted for twist-off-type tension control bolts of equal minimum specified tensile strength, and vice versa.

(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 or B slip resistance, or other) as that to be used for the corresponding bolts in the prototype.

4. Loading History

4a. General Requirements

The test specimen shall be subjected to cyclic loads according to the requirements prescribed in Section J2.4b for beam-to-column moment connections in special and intermediate moment frames, and according to the requirements prescribed in Section J2.4c for link-to-column connections in eccentrically braced frames.

Loading sequences other than those specified in Sections J2.4b and J2.4c may be used when they are demonstrated to be of equivalent or greater severity.

4b. Loading Sequence for Beam-to-Column Moment Connections

Qualifying cyclic tests of beam-to-column moment connections in special and intermediate moment frames shall be conducted by controlling the interstory drift angle, θ, imposed on the test specimen, as specified below:

(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 $\theta = 0.01$ rad, with two cycles of loading at each step.

4c. 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 *total link rotation angle*, γ_{total}, imposed on the test specimen, as follows:

(1) 6 cycles at $\gamma_{total} = 0.00375$ rad

(2) 6 cycles at $\gamma_{total} = 0.005$ rad

(3) 6 cycles at $\gamma_{total} = 0.0075$ rad

(4) 6 cycles at $\gamma_{total} = 0.01$ rad

(5) 4 cycles at $\gamma_{total} = 0.015$ rad

(6) 4 cycles at $\gamma_{total} = 0.02$ rad

(7) 2 cycles at $\gamma_{total} = 0.03$ rad

(8) 1 cycle at $\gamma_{total} = 0.04$ rad

(9) 1 cycle at $\gamma_{total} = 0.05$ rad

(10) 1 cycle at $\gamma_{total} = 0.07$ rad

(11) 1 cycle at $\gamma_{total} = 0.09$ rad

Continue loading at increments of γ_{total} = 0.02 rad, with one cycle of loading at each step.

5. Instrumentation

Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section J2.7.

6. Testing Requirements for Material Specimens

6a. Tension Testing Requirements for Structural Steel Material Specimens

Tension testing shall be conducted on samples taken from *material test plates* in accordance with Section J2.6b. The material test plates shall be taken from the steel of the same heat as used in the *test specimen*. Tension-test results from certified material test reports shall be reported, but shall not be used in lieu of physical testing for the purposes of this Section. 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.

6b. Methods of Tension Testing for Structural Steel Material Specimens

Tension testing shall be conducted in accordance with ASTM A6/A6M, ASTM A370, and ASTM E8, with the following exceptions:

(1) The yield strength, 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.

6c. Testing Requirements for Weld Metal Material Specimens

Weld metal testing shall be conducted on samples extracted from the *material test plate*, made using the same filler metal classification, manufacturer, brand or trade name and diameter, and using the same average heat input as used in the welding of the *test specimen*. The tensile strength and CVN toughness of weld material specimens shall be determined in accordance with *Standard Methods for Mechanical Testing of Welds* (ANSI/AWS B4.0). The use of tensile strength and CVN toughness values that are reported on the manufacturer's typical certificate of conformance in lieu of physical testing shall be prohibited for use for purposes of this section.

The same WPS shall be used to make the test specimen and the material test plate. The material test plate shall use base metal of the same grade and type as was used for the test specimen, although the same heat need not be used. If the average heat input used for making the material test plate is not within ±20% of that used for the test specimen, a new material test plate shall be made and tested.

7. 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 J2.3.

(4) A listing or plot showing the applied load or displacement history of the test specimen.

(5) A listing of all welds to be designated *demand critical*.

(6) Definition of the region of the member and connection to be designated a *protected zone.*

(7) 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.

(8) 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.

(9) 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.

(10) A chronological listing of significant test observations, including observations of yielding, slip, instability, and rupture of any portion of the test specimen as applicable.

(11) 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.

(12) The results of the material specimen tests specified in Section J2.6.

(13) 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.

8. Acceptance Criteria

The test specimen must satisfy the strength and *interstory drift angle* or link rotation angle requirements of these *Provisions* for the *special moment frame*, *intermediate moment frame*, or *eccentrically braced frame* connection, as applicable. The test specimen must sustain the required interstory drift angle or link rotation angle for at least one complete loading cycle.

J3. QUALIFYING CYCLIC TESTS OF BUCKLING-RESTRAINED BRACES

1. Scope

This section includes requirements for qualifying cyclic tests of individual buckling-restrained braces and buckling-restrained brace subassemblages, when required in these provisions. The purpose of the testing of individual braces is to provide evidence that a buckling-restrained brace satisfies the requirements for strength and inelastic deformation by these provisions; it also permits the determination of maximum brace forces for design of adjoining elements. The purpose of testing of the brace subassemblage is to provide evidence that the brace-design can satisfactorily accommodate the deformation and rotational demands associated with the design. Further, the subassemblage test is intended to demonstrate that the hysteretic behavior of the brace in the subassemblage is consistent with that of the individual brace elements tested uniaxially.

Alternative testing requirements are permitted when approved by the engineer of record and the *authority having jurisdiction.*
This section provides only minimum recommendations for simplified test conditions.

2. Subassemblage Test Specimen

The *subassemblage test specimen* shall satisfy the following requirements:

(1) The mechanism for accommodating inelastic rotation in the subassemblage test specimen brace shall be the same as that of the *prototype*. The rotational deformation demands on the subassemblage test specimen brace shall be equal to or greater than those of the prototype.

(2) The axial yield strength of the steel core, P_{ysc}, of the brace in the subassemblage test specimen shall not be less than that of the prototype where both strengths are based on the core area, A_{sc}, multiplied by the yield strength as determined from a coupon test.

(3) The cross-sectional shape and orientation of the steel core projection of the subassemblage test specimen brace shall be the same as that of the brace in the prototype.

(4) The same documented design methodology shall be used for design of the subassemblage as used for the prototype, to allow comparison of the rotational deformation demands on the subassemblage brace to the prototype. In stability calculations, beams, columns and gussets connecting the core shall be considered parts of this system.

(5) The calculated margins of safety for the prototype connection design, steel core projection stability, overall buckling and other relevant subassemblage test specimen brace construction details, excluding the gusset plate, for the prototype, shall equal or exceed those of the subassemblage test specimen construction.

(6) Lateral bracing of the subassemblage test specimen shall replicate the lateral bracing in the prototype.

(7) The *brace test specimen* and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

Extrapolation beyond the limitations stated in this section shall be permitted subject to qualified peer review and approval by the *authority having jurisdiction*.

3. Brace Test Specimen

The *brace test specimen* shall replicate as closely as is practical the pertinent design, detailing, construction features, and material properties of the *prototype*.

3a. Design of Brace Test Specimen

The same documented *design methodology* shall be used for the brace test specimen and the prototype. The design calculations shall demonstrate, at a minimum, the following requirements:

(1) The calculated margin of safety for stability against overall buckling for the prototype shall equal or exceed that of the brace test specimen.

(2) The calculated margins of safety for the brace test specimen and the prototype shall account for differences in material properties, including yield and ultimate stress, ultimate elongation, and toughness.

3b. Manufacture of Brace Test Specimen

The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

3c. Similarity of Brace Test Specimen and Prototype

The brace test specimen shall meet the following requirements:

(1) The cross-sectional shape and orientation of the steel core shall be the same as that of the prototype.

(2) The axial yield strength of the steel core, P_{ysc}, of the brace test specimen shall not be less than 50% nor more than 120% of the prototype where both strengths are based on the core area, A_{sc}, multiplied by the yield strength as determined from a coupon test.

(3) The material for, and method of, separation between the steel core and the buckling restraining mechanism in the brace test specimen shall be the same as that in the prototype.

Extrapolation beyond the limitations stated in this section shall be permitted subject to qualified peer review and approval by the *authority having jurisdiction*.

3d. Connection Details

The connection details used in the brace test specimen shall represent the prototype connection details as closely as practical.

3e. Materials

(1) Steel Core

The following requirements shall be satisfied for the steel core of the brace test specimen:

(a) The specified minimum yield stress of the brace test specimen steel core shall be the same as that of the prototype.

(b) The measured yield stress of the material of the steel core in the brace test specimen shall be at least 90% of that of the prototype as determined from coupon tests.

(c) The specified minimum ultimate stress and strain of the brace test specimen steel core shall not exceed those of the prototype.

(2) Buckling-Restraining Mechanism

Materials used in the *buckling-restraining mechanism* of the brace test specimen shall be the same as those used in the prototype.

3f. Connections

The welded, bolted, and pinned joints on the test specimen shall replicate those on the prototype as close as practical.

4. Loading History

4a. General Requirements

The *test specimen* shall be subjected to cyclic loads according to the requirements prescribed in Sections J3.4b and J3.4c. Additional increments of loading beyond those described in Section J3.4c are permitted. Each cycle shall include a full tension and full compression excursion to the prescribed deformation.

4b. Test Control

The test shall be conducted by controlling the level of axial or rotational deformation, Δ_b, imposed on the test specimen. As an alternate, the maximum rotational deformation may be applied and maintained as the protocol is followed for axial deformation.

4c. Loading Sequence

Loads shall be applied to the test specimen to produce the following deformations, where the deformation is the steel core axial deformation for the test specimen and the rotational deformation demand for the *subassemblage test specimen* brace:

(1) 2 cycles of loading at the deformation corresponding to $\Delta_b = \Delta_{by}$.

(2) 2 cycles of loading at the deformation corresponding to $\Delta_b = 0.50\ \Delta_{bm}$.

(3) 2 cycles of loading at the deformation corresponding to $\Delta_b = 1\ \Delta_{bm}$.

(4) 2 cycles of loading at the deformation corresponding to $\Delta_b = 1.5\ \Delta_{bm}$.

(5) 2 cycles of loading at the deformation corresponding to $\Delta_b = 2.0\ \Delta_{bm}$.

(6) Additional complete cycles of loading at the deformation corresponding to $\Delta_b = 1.5\Delta_{bm}$ as required for the brace test specimen to achieve a cumulative inelastic axial deformation of at least 200 times the yield deformation (not required for the subassemblage test specimen).

The *design story drift* shall not be taken as less than 0.01 times the story height for the purposes of calculating Δ_{bm}. Other loading sequences are permitted to be used to qualify the test specimen when they are demonstrated to be of equal or greater severity in terms of maximum and cumulative inelastic deformation.

5. Instrumentation

Sufficient instrumentation shall be provided on the *test specimen* to permit measurement or calculation of the quantities listed in Section J3.7.

6. Materials Testing Requirements

6a. Tension Testing Requirements

Tension testing shall be conducted on samples of steel taken from the same heat of steel as that used to manufacture the steel core. Tension test results from certified material test reports shall be reported but are prohibited in place of material specimen testing for the purposes of this Section. Tension-test results shall be based upon testing that is conducted in accordance with Section J3.6b.

6b. Methods of Tension Testing

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

(1) The yield stress that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method of 0.002 strain.

(2) The loading rate for the tension test shall replicate, as closely as is practical, the loading rate used for the *test specimen*.

(3) The coupon shall be machined so that its longitudinal axis is parallel to the longitudinal axis of the steel core.

7. Test Reporting Requirements

For each *test specimen*, a written test report meeting 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 specimen, including key dimensions, boundary conditions at loading and reaction points, and location of lateral bracing, if any.

(2) A drawing of the connection details showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt or pin holes, the size and grade of connectors, and all other pertinent details of the connections.

(3) A listing of all other essential variables as listed in Sections J3.2 or J3.3, as appropriate.

(4) A listing or plot showing the applied load or displacement history.

(5) A plot of the applied load versus the deformation, Δ_b. The method used to determine the deformations shall be clearly shown. The

locations on the *test specimen* where the loads and deformations were measured shall be clearly identified.

(6) A chronological listing of significant test observations, including observations of yielding, slip, instability, transverse displacement along the test specimen and rupture of any portion of the test specimen and connections, as applicable.

(7) The results of the material specimen tests specified in Section J3.6.

(8) The manufacturing quality control and quality assurance plans used for the fabrication of the test specimen. These shall be included with the welding procedure specifications and welding inspection reports.

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

8. Acceptance Criteria

At least one subassemblage test that satisfies the requirements of Section J3.2 shall be performed. At least one brace test that satisfies the requirements of Section J3.3 shall be performed. Within the required protocol range all tests shall satisfy the following requirements:

(1) The plot showing the applied load vs. displacement history shall exhibit stable, repeatable behavior with positive incremental stiffness.

(2) There shall be no rupture, brace instability or brace end connection failure.

(i) For brace tests, each cycle to a deformation greater than Δ_{by} the maximum tension and compression forces shall not be less than the nominal strength of the core.

(ii) For brace tests, each cycle to a deformation greater than Δ_{by} the ratio of the maximum compression force to the maximum tension force shall not exceed 1.3.

Other acceptance criteria may be adopted for the *brace test specimen* or *subassemblage test specimen* subject to qualified peer review and approval by the *authority having jurisdiction.*