Seismic Provisions for Structural Steel Buildings

Including Supplement No. 1

April 15, 1997

Including Supplement No. 1 (February 15, 1999)

Note: This is an interim version that incorporates Supplement 1 into the 1997 document. This version will not be available in print, but will be available for free download on the AISC web site.

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approved by the AISC Board of Directors.

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PREFACE

This **interim version** of the AISC *Seismic Provisions for Structural Steel Buildings* incorporates Supplement No. 1 (February 19, 1999). A sidebar is shown in the outside margins where revisions have occurred based on this supplement. Note: The formatting and page numbers are not as they appear in the printed version of the 1997 *Seismic Provisions for Structural Steel Buildings*.

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

The AISC Committee on Specification, Task Committee 9—Seismic Provisions is responsible for ongoing development of these Provisions. Additionally, the AISC Committee on Specification has enhanced these Provisions through careful scrutiny, discussion, suggestion for improvements, and endorsement. AISC further acknowledges the various contributions of several groups to the completion of this document: the Building Seismic Safety Council (BSSC), the National Science Foundation (NSF), the SAC Joint Venture, and the Structural Engineers Association of California (SEAOC).

Supplement Number 1 to the 1997 AISC *Seismic Provisions for Structural Steel Buildings* includes revisions to the requirements for panel zone shear strength in Special Moment Frames (SMF's) and tightens the column width-thickness ratio and lateral bracing requirements for conditions where column inelasticity is a possibility. Consistent changes were made in Part II of the document and also in the Commentary, along with a few other improvements.

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Symbols

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

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

Required axial strength of a composite column, kips. (II-5)

P_{uc} Required axial strength on a column in compression, kips. (I-9)

P_y Nominal axial yield strength of a member, which is equal to F_yA_g, kips. (I-9)

Q_b Maximum unbalanced vertical load effect applied to a beam by the braces, kips. (I-13)

Q_E Effect of horizontal seismic forces produced by the base shear, V. (I-4)

R_n Nominal strength. (I-9)

R_u Required strength. (I-9)

R_y Ratio of the Expected Yield Strength F_{ye} to the minimum specified yield strength F_y. (I-5)

S Snow load, kips. (I-4)

S_a Design spectral response acceleration. (I-4)

V_n Nominal shear strength of a member, kips. (I-9)

V_{ns} Nominal shear strength of the steel plate in a composite plate shear walls, kips. (II-5)

V_p Nominal shear strength of an active Link, kips. (I-15)

V_{pa} Nominal shear strength of an active Link modified by the axial load magnitude, kips. (I-15)

V_u Required shear strength on a member, kips. (I-9)

Y_{con} Distance from top of steel beam to top of concrete slab or encasement, in. (II-6)

Z Plastic section modulus of a member, in.^{3} (I-9)

a Angle that diagonal members make with the horizontal. (I-12)

b Width of compression element as defined in LRFD Specification Section B5.1, in. (Table I-9-1)

b_{cf} Width of column flange, in. (I-9)

b_f Flange width, in. (I-9)

b_w Width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear, in. (II-6)

d Nominal fastener diameter, in. (I-7)

d_b Overall beam depth, in. (I-9)

d_c Overall column depth, in. (I-9)

d_z Overall panel-zone depth between continuity plates, in. (I-9)

e EBF Link length, in. (I-15)

f'_c Specified compressive strength of concrete, ksi. (II-6)

h Cross-sectional dimension of reinforced concrete or composite column, in. (II-6)

h_c Assumed web depth for stability, in. (Table I-9-1)

h_{cc} Cross-sectional dimension of the confined core region in composite columns measured center-to-center of the transverse reinforcement., in. (II-6)

l unbraced length between stitches of built-up bracing members, in. (I-13)

r Governing radius of gyration, in. (I-13)

r_y Radius of gyration about y axis, in. (I-9)

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

t Thickness of connected part, in. (I-7)

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

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

t_f Thickness of flange, in. (Table I-9-1)

t_p Thickness of panel-zone including doubler plates, in. (I-9)

t_w Thickness of web, in. (Table I-9-1)

t_z Thickness of panel-zone (doubler-plate thickness not necessarily included), in. (I-9)

w_z Width of panel-zone between column flanges, in. (I-9)

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

Δ Design story drift. in. (I-6)

ΣM^*_{pc} Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g, from the top and bottom of the beam moment connection. (I-9)

ΣM^*_{pb} Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum developed moments from the column face. Maximum developed moments shall be determined from test results. (I-9)

Ω_o Horizontal seismic overstrength factor. (I-4)

δ Deformation quantity used to control loading of the Test Specimen. (S6)

δ_y Value of deformation quantity δ at first significant yield of Test Specimen. (S6)

ρ' Ratio of required axial force P_u to required shear strength V_u of a Link. (I-15)

λ Slenderness parameter. (I-13)

λ_p Limiting slenderness parameter for compact element. (Table I-9-1)

λ_r Limiting slenderness parameter for non-compact element. (I-14)

ϕ Resistance factor. (I-8)

ϕ_c Resistance factor for compression. (I-13)

ϕ_v Resistance factor for shear strength of panel-zone of beam-to-column connections. (I-9)

ϕ_v Resistance factor for the shear strength of a composite column. (II-6)

ρ_v Ratio of distributed vertical or horizontal reinforcement to the gross wall area. (II-5)

Part I
Structural Steel Buildings

Part I Glossary

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Joint. An area where two or more ends, surfaces or edges are attached. Joints are categorized by the type of fastener or weld used and the method of force transfer.

K-Braced Frame. An OCBF in which a pair of diagonal braces located on one side of a column is connected to a single point within the clear column height.

Lateral Support Member. A member that is designed to inhibit lateral buckling or lateral-torsional buckling of primary framing members.

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

Link Intermediate Web Stiffeners. Vertical web stiffeners placed within the Link in EBF.

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

Link Shear Design Strength. The lesser of the design shear strength of the Link developed from the moment or shear strength of the Link.

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

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

Nominal loads. The magnitudes of the loads specified by the Applicable Building Code.

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

Ordinary Concentrically Braced Frame (OCBF). A diagonally braced frame meeting the requirements in Section 14 in which all members of the bracing system are subjected primarily to axial forces.

Ordinary Moment Frame (OMF). A moment frame system that meets the requirements in Section 11.

P - Delta Effect. Second-order effect of column axial loads after lateral deflection of the frame on the shears and moments in members.

Panel-zone. The web area of the beam-to-column connection delineated by the extension of beam and column flanges through the connection.

Partially Restrained (PR). Insufficient rigidity exists in the connection to maintain the angles between intersecting members.

Reduced Beam Section. A reduction in cross-section over a discrete length that promotes a zone of inelasticity in the member.

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

Resistance Factor. A factor that accounts for unavoidable deviations in the actual strength of a member or connection from the Nominal Strength and for the manner and consequences of failure.

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

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

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

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

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

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

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

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

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

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

Y-Braced Frame. An Eccentrically Braced Frame (EBF) in which the stem of the Y is the Link of the EBF system.

1. SCOPE

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

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

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

2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

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

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

American Society of Civil Engineers
ASCE 7-95

American Society for Testing and Materials

ASTMA6-96b	ASTM A500-93	ASTM A673-95
ASTM A36-96	ASTM A501-93	ASTM A913-95a
ASTM A53-96	ASTM A572-94c	ASTM A992-98
ASTM A283-93a	ASTM A588-94	

American Welding Society
AWS D1.1-96

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

3. SEISMIC DESIGN CATEGORIES

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

4. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

4.1. Loads and Load Combinations

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

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

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

$$1.2D + 0.5L + 0.2S + \Omega_o Q_E \tag{4-1}$$

$$0.9D - \Omega_o Q_E \tag{4-2}$$

Exception: The load factor on L in Load Combination 4-1 shall equal 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.

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

TABLE I-4-1
System Overstrength Factor, Ω_o

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

4.2 Nominal Strength

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

5. STORY DRIFT

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

6. MATERIALS

6.1. Material Specifications

Structural steel used in the Seismic Force Resisting System shall meet the requirements in LRFD Specification Section A3.1a, except as modified in this Section. For buildings over one story in height, the steel used in the Seismic Force Resisting Systems described in Sections 9, 10, 11, 12, 13, 14 and 15 shall meet one of the following ASTM Specifications: A36, A53, A500 (Grade B or C), A501, A572 (Grade 42 or 50), A588,A913 (Grade 50 or 65), or A992. The steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283 Grade D. The specified minimum yield strength of steel to be used for members in which inelastic behavior is expected under Load Combinations 4-1 and 4-2 shall not exceed 50 ksi unless the suitability of the material is determined by testing or other rational criteria. This limitation does not apply to columns for which the only expected inelastic behavior is yielding at the column base.

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

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

$$F_{ye} = R_y F_y \tag{6-1}$$

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

6.3. Notch-Tough Steel

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

7. CONNECTIONS, JOINTS AND FASTENERS

7.1. Scope

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

7.2. Bolted Joints

7.2a. All bolts shall be fully tensioned high-strength bolts. All faying surfaces shall be prepared as required for Class A or better slip-critical joints. The design shear strength of bolted joints is permitted to be calculated as that for bearing-type joints.

7.2b. Bolted joints shall not be designed to share load in combination with welds on the same faying surface.

7.2c. The bearing strength of bolted joints shall be provided using either standard holes or short-slotted holes with the slot perpendicular to the line of force, unless an alternative hole type is justified as part of a tested assembly; see Appendix S.

7.2d. The design strength of bolted joints in shear and/or combined tension and shear shall be determined in accordance with LRFD Specification Sections J3.7 and J3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

7.2e. Bolted connections for members that are a part of the Seismic Force Resisting System shall be configured such that a ductile limit-state either in the connection or in the member controls the design.

7.3. Welded Joints

7.3a. Welding shall be performed in accordance with a Welding Procedure Specification (WPS) as required in AWS D1.1 and approved by the Engineer of Record. The WPS variables shall be within the parameters established by the filler-metal manufacturer.

7.3b. All welds used in primary members and connections in the Seismic Force Resisting System shall be made with a filler metal that has a minimum Charpy V-Notch toughness of 20 ft-lbs at minus 20 degrees F, as determined by AWS classification or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these Provisions.

7.3c. For members and connections that are part of the Seismic Force Resisting System, discontinuities created by errors or by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging, and flame cutting, shall be repaired as required by the Engineer of Record.

8. COLUMNS

8.1. Scope

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

8.2. Column Strength

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

8.2a. The required axial compressive strength, considered in the absence of any applied moment, shall be determined from Load Combination 4-1.

8.2b. The required axial tensile strength, considered in the absence of any applied moment, shall be determined from Load Combination 4-2.

8.2c. The required strengths determined in Sections 8.2a and 8.2b need not exceed either of the following:

a. The maximum load transferred to the column considering $1.1R_y$ times the nominal strengths of the connecting beam or brace elements of the building.

b. The limit as determined from the resistance of the foundation to overturning uplift.

8.3. Column Splices

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

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

1. The design strength of partial-joint-penetration groove welded joints shall be at least equal to 200 percent of the required strength.

2. The minimum required strength shall be 0.5 times $R_yF_yA_f$, where R_yF_y is the Expected Yield Strength of the column material and A_f is the flange area of the smaller column connected.

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

9. SPECIAL MOMENT FRAMES (SMF)

9.1. Scope

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

9.2. Beam-to-Column Joints and Connections

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

a. Tests reported in research or documented tests performed for other projects that are demonstrated to reasonably match project conditions.

b. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes.

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

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

9.2b. Beam-to-column connection testing shall demonstrate a flexural strength, determined at the column face, that is at least equal to the nominal plastic moment of the beam M_p at the required inelastic rotation (see Appendix S), except as follows:

a. When beam local buckling rather than beam yielding limits the flexural strength of the beam, or when connections incorporating a Reduced Beam Section are used, the minimum flexural strength shall be $0.8M_p$ of the tested beam.

b. Connections that accommodate the required rotations within the connecting elements and maintain the design strength as specified in Section 1 are permitted, provided it can be demonstrated by rational analysis that any additional drift due to connection deformation can be accommodated by the building. Such rational analysis shall include the effects of overall frame stability including second-order effects.

9.2c. The required shear strength V_u of a beam-to-column connection shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from the application of $1.1R_yF_yZ$ in the opposite sense on each end of the beam. Alternatively, a lesser value of V_u is permitted if justified by rational analysis. The required shear strength need not exceed the shear resulting from Load Combination 4-1.

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

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

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

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

t_p = total thickness of panel-zone including doubler plate(s), in.

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

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

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

where

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

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

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

9.4. Beam and Column Limitations

9.4a. Beam Flange Area: Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is permitted if testing demonstrates that the resulting configuration can develop stable plastic hinges that meet the requirements in Section 9.2b. The Reduced Beam Section shall meet the design strength as specified in Section 1.

9.4b. Width-Thickness Ratios: Beams shall comply with λ_p in Table I-9-1. When the ratio in Equation 9-3 is less than or equal to 1.25, columns shall comply with λ_p in Table I-9-1. Otherwise, columns shall comply with λ_p in LRFD Specification Table B5.1.

TABLE I-9-1
Limiting Width Thickness Ratios λ_p
for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios λ_p
Flanges of I-shaped rolled beams, hybrid or welded beams and channels in flexure	b/t	$52/\sqrt{F_y}$
Webs in combined flexural and axial compression	h/t_w	For $P_u/\phi_b P_y \leq 0.125$
		$\dfrac{520}{\sqrt{F_y}}\left[1-1.54\dfrac{P_u}{\phi_b P_y}\right]$
		For $P_u/\phi_b P_y > 0.125$
		$\dfrac{191}{\sqrt{F_y}}\left[2.33-\dfrac{P_u}{\phi_b P_y}\right] \geq \dfrac{253}{\sqrt{F_y}}$
Round HSS in axial compression or flexure	D/t	$1300/F_y$
Rectangular HSS in axial compression or flexure	b/t or h_c/t	$110/\sqrt{F_y}$

9.5. Continuity Plates

Continuity plates shall be provided to match the tested connection.

9.6. Column-Beam Moment Ratio

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

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

where

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

ΣM^*_{pb}	=	The sum of the moment(s) in the beam(s) at the intersection of the beam and column centerlines. ΣM^*_{pb} is determined by summing the projections of the expected beam flexural strength(s) at the plastic hinge location(s) to the column centerline. It is permitted to take $\Sigma M^*_{pb} = \Sigma(1.1R_yM_p+M_v)$, where M_v is the additional moment due to shear amplification from the location of the plastic hinge to the column centerline. Alternatively, it is permitted to determine ΣM^*_{pb} from test results as required in Section 9.2a or by rational analysis based upon the tests. When connections with Reduced Beam Sections are used, it is permitted to take $\Sigma M^*_{pb} = \Sigma(1.1R_yF_yz+M_v)$, where z is the minimum plastic section modulus at the Reduced Beam Section.
A_g	=	gross area of column, in.^{2}
F_{yc}	=	specified minimum yield strength of column, ksi
P_{uc}	=	required column axial compressive strength, kips (a positive number)
Z_c	=	plastic section modulus of the column, in.^{3}

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

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

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

b. Columns where: (1) the sum of the design shear strengths of all exempted columns in the story is less than 20 percent of the required story shear strength; and (2) the sum of the design shear strengths of all exempted columns on each column line within that story is less than 33 percent of the required story shear strength on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

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

9.7. Beam-to-Column Connection Restraint

9.7a. Restrained Connections:

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

   a. The ratio calculated using Equation 9-3 is greater than 1.25.
   b. The column remains elastic under Load Combination 4-1.

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

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

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

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

9.7b. Unrestrained Connections: A column containing a beam-to-column connection with no lateral support transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral supports as the column height for buckling transverse to the seismic frame and shall conform to LRFD Specification Chapter H, except that:

1. The required column strength shall be determined from LRFD Specification Load Combination A4-5, except that E shall be taken as the lesser of:

   a. The amplified earthquake force $\Omega_o Q_E$.

   b. 125 percent of the frame design strength based upon either the beam design flexural strength or panel-zone design shear strength.

2. The slenderness L/r for the column shall not exceed 60.

3. The column required flexural strength transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Section 9.7a.2.b in addition to the second-order moment due to the resulting column flange displacement.

9.8. Lateral Support of Beams

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

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

10. INTERMEDIATE MOMENT FRAMES (IMF)

10.1. Scope

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

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

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

10.2. Beam-to-Column Joints and Connections

10.2a. The design of all beam-to-column joints and connections used in the Seismic Force Resisting System shall be based upon qualifying cyclic test results in accordance with Appendix S that demonstrate an inelastic rotation of at least 0.02 radians. Qualifying cyclic tests results shall consist of at least two cyclic tests and shall meet the requirements in Section 9.2a.

10.2b. Beam-to-column connection testing shall demonstrate a flexural strength, determined at the column face, that is at least equal to the nominal plastic moment of the beam M_p at the required inelastic rotation (see Appendix S), except as follows:

a. When beam local buckling rather than beam yielding limits the flexural strength of the beam, or when connections incorporating a Reduced Beam Section are used, the minimum flexural strength shall be $0.8M_p$ of the tested beam.

b. Connections that accommodate the required rotations within the connection elements and maintain the design strength as specified in Section 1 are permitted, provided it can be demonstrated by rational analysis that any additional drift due to connection deformation can be accommodated by the building. Such rational analysis shall include the effects of overall frame stability including second order effects.

Replace Section 9.4b with 10.4b as follows:

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

Replace Section 9.8 with 10.8 as follows:

10.8. Lateral Support at Beams

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

11. ORDINARY MOMENT FRAMES (OMF)

11.1. Scope

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

11.2. Beam-to-Column Joints and Connections

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

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

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

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

   1. Such connections shall provide for the design strength as specified in Section 1.

   2. The nominal flexural strength of the connection shall be equal to or exceed 50 percent of M_p of the connected beam or column, whichever is less.

   3. Adequate rotation capacity shall be demonstrated in the connections by cyclic testing at rotations corresponding to the Design Story Drift.

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

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

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

11.3. Continuity Plates

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

a. The sum of the design strengths at the connections of the continuity plate to the column flanges.

b. The design shear strength of the contact area of the plate with the column web.

c. The weld design strength that develops the design shear strength of the column panel-zone.

d. The actual force transmitted by the stiffener.

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

12. SPECIAL TRUSS MOMENT FRAMES (STMF)

12.1. Scope

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

12.2. Special Segment

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

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

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

12.3. Nominal Strength of Special Segment Members

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

12.4. Nominal Strength of Non-special Segment Members

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

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

where

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

12.5. Compactness

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

12.6. Lateral Bracing

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

13. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

13.1. Scope

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

13.2. Bracing Members

13.2a. Slenderness: Bracing members shall have $Kl/r \leq 1000/\sqrt{F_y}$.

13.2b. Required Compressive Strength: The required strength of a bracing member in axial compression shall not exceed $\phi_c P_n$.

13.2c. Lateral Force Distribution: Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force is resisted by tension braces, unless the nominal strength P_n of each brace in compression is larger than the required strength P_u resulting from the application of Load Combinations 4-1 or 4-2. For the purposes of this provision, a line of bracing is defined as a single line or parallel lines whose plan offset is 10 percent or less of the building dimension perpendicular to the line of bracing.

13.2d. Width-thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements of braces shall meet the compactness requirements in LRFD Specification Table B5.1 (i.e. $\lambda<\lambda_p$) and the following requirements:

1. The width-thickness ratio of angles shall not exceed $52/\sqrt{F_y}$.
2. I-shaped members and channels used as braces shall comply with λ_p in Table I-9-1.
3. Round HSS shall have an outside diameter to wall thickness ratio conforming to Table I-9-1 unless the round HSS wall is stiffened.
4. Rectangular HSS shall have a flat width to wall thickness ratio conforming to Table I-9-1 unless the rectangular HSS walls are stiffened.

13.2e. Built-up Members: The spacing of stitches shall be such that the slenderness ratio l/r of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member.

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

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

13.3. Bracing Connections

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

a. The nominal axial tensile strength of the bracing member, determined as $R_yF_yA_g$.

b. The maximum force, indicated by analysis, that can be transferred to the brace by the system.

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

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

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

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

13.4. Special Bracing Configuration Special Requirements

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

1. A beam that is intersected by braces shall be continuous between columns.

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

3. A beam that is intersected by braces shall be designed to resist the effects of LRFD Specification Load Combinations A4-5 and A4-6 except that a load Q_b shall be substituted for the term E. Q_b is the maximum unbalanced vertical load effect applied to the beam by the braces. This load effect shall be calculated using a minimum of P_y for the brace in tension and a maximum of 0.3 times ϕ_cP_n for the brace in compression.

4. The top and bottom flanges of the beam at the point of intersection of braces shall be designed to support a lateral force that is equal to 2 percent of the nominal beam flange strength $F_yb_ft_{bf}$.

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

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

13.5. Columns

Columns in SCBF shall meet the following requirements:

13.5a. Width-thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements of columns shall meet the requirements for bracing members in Section 13.2d.

13.5b. Splices: In addition to meeting the requirements in Section 8.3, column splices in SCBF shall be designed to develop at least the nominal shear strength of the smaller connected member and 50 percent of the nominal flexural strength of the smaller connected section. Splices shall be located in the middle one-third of the column clear height.

14. ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

14.1. Scope

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

14.2. Bracing Members

14.2a. Slenderness: Bracing members shall have $Kl/r \leq 720/\sqrt{F_y}$ except as permitted in Section 14.5.

14.2b. Required Compressive Strength: The required strength of a bracing member in axial compression shall not exceed 0.8 times $\phi_c P_n$.

14.2c. Lateral Force Distribution: Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force is resisted by tension braces, unless the nominal strength P_n of each brace in compression is larger than the required strength P_u resulting from the application of Load Combinations 4-1 or 4-2. A line of bracing, for the purposes of this provision, is defined as a single line or parallel lines whose plan offset is 10 percent or less of the building dimension perpendicular to the line of bracing.

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

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

9-1 unless the rectangular section walls are stiffened.

14.2e. Built-up Member Stitches: For all built-up braces, the first bolted or welded stitch on each side of the mid-length of a built up member shall be designed to transmit a force equal to 50 percent of the nominal strength of one element to the adjacent element. Not less than two stitches shall be equally spaced about the member centerline.

14.3. Bracing Connections

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

a. The nominal axial tensile strength of the bracing member, determined as $R_yF_yA_g$.

b. The force in the brace that results from Load Combinations 4-1 and 4-2.

c. The maximum force, indicated by analysis, that can be transferred to the brace by the system.

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

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

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

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

14.4. Bracing Configuration Special Requirements

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

1. The design strength of brace members shall be at least 1.5 times the required strength using LRFD Specification Load Combinations A4-5 and A4-6.

2. A beam that is intersected by braces shall be continuous between columns.

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

4. The top and bottom flanges of the beam at the point of intersection of braces shall be designed to support a lateral force that is equal to 2 percent of the nominal beam flange strength $F_y b_f t_f$.

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

14.5. Low Buildings

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

15. ECCENTRICALLY BRACED FRAMES (EBF)

15.1. Scope

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

15.2. Links

15.2a. Links shall comply with the width-thickness ratios in Table I-9-1.

15.2b. The specified minimum yield stress of steel used for Links shall not exceed 50 ksi.

15.2c. The web of a Link shall be single thickness without doubler-plate reinforcement and without web penetrations.

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

V_n = nominal shear strength of the Link, equal to the lesser of V_p or $2M_p/e$, kips.
V_p = $0.6F_y(d-2t_f)\, t_w$, kips.
e = Link length, in.
ϕ = 0.9.

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

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

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

$$\phi = 0.9$$

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

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

2. The length of the Link shall not exceed:

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

nor

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

where:

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

15.2g. The Link Rotation Angle is the inelastic angle between the Link and the beam outside of the Link when the total story drift is equal to the Design Story Drift, Δ. The Link Rotation Angle shall not exceed the following values:

a. 0.08 radians for Links of length $1.6M_p/V_p$ or less.

b. 0.02 radians for Links of length $2.6M_p/V_p$ or greater.

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

15.3. Link Stiffeners

15.3a. Full-depth web stiffeners shall be provided on both sides of the Link web at the diagonal brace ends of the Link. These stiffeners shall have a combined width not less than (b_f - $2t_w$) and a thickness not less than $0.75t_w$ nor 3/8 in., whichever is larger, where b_f and t_w are the Link flange width and Link web thickness, respectively.

15.3b. Links shall be provided with intermediate web stiffeners as follows:

1. Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding ($30t_w$-d/5) for a Link Rotation Angle of 0.08 radians or ($52t_w$-d/5) for Link Rotation Angles of 0.02 radians or less. Linear interpolation shall be used for values between 0.08 and 0.02 radians.

2. Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the Link.

3. Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of 1 and 2 above.

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

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

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

15.4. Link-to-Column Connections

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

15.4a. The Link-to-column connection design shall be based upon cyclic test results that demonstrate an inelastic rotation capability that is 20 percent greater than that calculated at the Design Story Drift, Δ. Qualifying test results shall be as described in Sections 9.2a and 9.2b., except that the inelastic rotation angle shall be as described in Section 15.2g.

15.4b. Where reinforcement at the beam-to-column connection at the Link end precludes yielding of the beam over the reinforced length, the Link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such Links are used and the Link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the design strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened Link as described in Section 15.6a. Full depth stiffeners as required in Section 15.3a. shall be placed at the Link-to-reinforcement interface.

15.5. Lateral Support of Link

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

15.6. Diagonal Brace and Beam Outside of Link

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

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

1. The required strength of the beam outside of the Link shall be the forces generated by at least 1.1 times the expected nominal shear strength of the Link R_yV_n, where V_n is as defined in Section 15.2. For determining the design strength of this portion of the beam, it is permitted to multiply the design strengths determined from the LRFD Specification

by R_y.

2. The beam shall be provided with lateral support where analysis indicates that support is necessary to maintain the stability of the beam. Lateral support shall be provided at both the top and bottom flanges of the beam and each shall have a required strength of at least 2 percent of the beam flange nominal strength computed as $F_y b_f t_f$.

15.6c. At the connection between the diagonal brace and the beam at the Link end of the brace, the intersection of the brace and beam centerlines shall be at the end of the Link or in the Link.

15.6d. The required strength of the diagonal brace-to-beam connection at the Link end of the brace shall be at least the expected nominal strength of the brace as given in Section 15.6a. No part of this connection shall extend over the Link length. If the brace resists a portion of the Link end moment, the connection shall be designed as an FR moment connection.

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

15.7. Beam-to-Column Connections

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

15.8. Required Column Strength

In addition to the requirements in Section 8, the required strength of columns shall be determined from LRFD Specification Load Combinations A4-5 and A4-6, except that the moments and axial loads introduced into the column at the connection of a Link or brace shall not be less than those generated by the expected nominal strength of the Link multiplied by 1.1 to account for strain-hardening. The expected nominal strength of the Link is $R_y V_n$, where V_n is as defined in Section 15.2d.

16. QUALITY ASSURANCE

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

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

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

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

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

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

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

Appendix S

QUALIFYING CYCLIC TESTS OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

S1. SCOPE AND PURPOSE

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

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

S2. SYMBOLS

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

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

S3. DEFINITIONS

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

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

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

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

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

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

S4. TEST SUBASSEMBLAGE REQUIREMENTS

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

1. The Test Specimen shall consist of at least a single column with beams or Links attached to one or both sides of the column.

2. Points of inflection in the test assemblage shall coincide approximately with the anticipated points of inflection in the Prototype under earthquake loading.

3. Lateral bracing of the Test Subassemblage is permitted near load application or reaction points as needed to provide lateral stability of the Test Subassemblage. Additional lateral bracing of the Test Subassemblage is not permitted, unless it replicates lateral bracing to be used in the Prototype.

S5. ESSENTIAL TEST VARIABLES

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

S5.1. Sources of Inelastic Rotation

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

S5.2. Size of Members

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

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

   b. The weight per foot of the test beam or Link shall be no less than 75 percent of the weight per foot of the Prototype beam or Link.

2. The size of the column used in the Test Specimen shall properly represent the inelastic action in the column, as per the requirements in Section S5.1

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

S5.3. Connection Details

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

S5.4. Continuity Plates

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

S5.5. Material Strength

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

1. The yield stress shall be determined by material tests on the actual materials used for the Test Specimen, as specified in Section S8. The use of yield stress values that are reported on certified mill test reports are not permitted to be used for purposes of this Section.

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

S5.6. Welds

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

1. Welding shall be performed in strict conformance with a Welding Procedure Specifications (WPS) as required in AWS D1.1. The WPS essential variables shall meet the requirements in AWS D1.1 and shall be within the parameters established by the filler-metal manufacturer.

2. The specified minimum tensile strength of the filler metal used for the Test Specimen shall be the same as that to be used for the corresponding Prototype welds.

3. The specified minimum CVN toughness of the filler metal used for the Test Specimen shall not exceed the specified minimum CVN toughness of the filler metal to be used for the corresponding Prototype welds.

4. The welding positions used to make the welds on the Test Specimen shall be the same as those to be used for the Prototype welds.

5. Details of weld backing, weld tabs, access holes, and similar items used for the Test Specimen welds shall be the same as those to be used for the corresponding Prototype welds. Weld backing and weld tabs shall not be removed from the Test Specimen welds unless the corresponding weld backing and weld tabs are removed from the Prototype welds.

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

S5.7. Bolts

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

1. The bolt grade (e.g., ASTM A325, ASTM A490) used in the Test Specimen shall be the same as that to be used for the Prototype.

2. The type and orientation of bolt holes (standard, oversize, short slot, long slot, or other) used in the Test Specimen shall be the same as those to be used for the corresponding bolt holes in the Prototype.

3. When Inelastic Rotation is to be developed either by yielding or by slip within a bolted portion of the connection, the method used to make the bolt holes (drilling, sub-punching and reaming, or other) in the Test Specimen shall be the same as that to be used in the corresponding bolt holes in the Prototype.

4. Bolts in the Test Specimen shall have the same installation (fully-tensioned or other) and faying surface preparation (no specified slip resistance, Class A slip resistance, or other) as that to be used for the corresponding bolts in the Prototype.

S6. LOADING HISTORY

S6.1. General Requirements

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

S6.2. Test Control

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

S6.3. Loading Sequence

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

1. 3 cycles of loading at: $0.25\delta_y < \delta \leq 0.5\delta_y$

2. 3 cycles of loading at: $0.6\delta_y < \delta \leq 0.8\delta_y$

3. 3 cycles of loading at: $\delta = \delta_y$

4. 3 cycles of loading at: $\delta = 2\delta_y$

5. 3 cycles of loading at: $\delta = 3\delta_y$

6. 2 cycles of loading at: $\delta = 4\delta_y$

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

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

S7. INSTRUMENTATION

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

S8. MATERIALS TESTING REQUIREMENTS

S8.1. Tension Testing Requirements

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

1. Flange(s) and web(s) of beams and columns at standard locations.

2. Any element of the connection that supplies Inelastic Rotation by yielding.

S8.2. Methods of Tension Testing

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

1. The yield stress F_y that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method at 0.002 strain.

2. The loading rate for the tension test shall replicate, as closely as practical, the loading rate to be used for the Test Specimen.

S9. TEST REPORTING REQUIREMENTS

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

1. A drawing or clear description of the Test Subassemblage, including key dimensions, boundary conditions at loading and reaction points, and location of lateral braces.

2. A drawing of the connection detail showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt holes, the size and grade of bolts, and all other pertinent details of the connection.

3. A listing of all other Essential Variables for the Test Specimen, as listed in Section S5.

4. A listing or plot showing the applied load or displacement history of the Test Specimen.

5. A plot of the applied load versus the displacement of the Test Specimen. The displacement reported in this plot shall be measured at or near the point of load application. The locations on the Test Specimen where the loads and displacements were measured shall be clearly indicated.

6. A plot of beam moment versus total Inelastic Rotation. The beam moment and the total Inelastic Rotation shall be computed with respect to the face of the column.

7. The total Inelastic Rotation developed by the Test Specimen. The components of the Test Specimen contributing to the total Inelastic Rotation due to yielding or slip shall be identified. The portion of the total Inelastic Rotation contributed by each component of the Test Specimen shall be reported. The method used to compute Inelastic Rotations shall be clearly shown.

8. A chronological listing of significant test observations, including observations of yielding, slip, instability, and fracture of any portion of the Test Specimen as applicable.

9. The controlling failure mode for the Test Specimen. If the test is terminated prior to failure, the reason for terminating the test shall be clearly indicated.

10. The results of the material tests specified in Section S8.

11. The Welding Procedure Specifications (WPS) and welding inspection reports.

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

S10. ACCEPTANCE CRITERIA

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

Part II—Composite Structural Steel and Reinforced Concrete Buildings

Part II Glossary

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Face Bearing Plates. Stiffeners that are attached to structural steel beams that are embedded in reinforced concrete walls or columns. The plates are located at the face of the reinforced concrete to provide confinement and to transfer forces to the concrete through direct bearing.

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

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

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

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

Partially Restrained Composite Connection. Partially restrained connections as defined in the LRFD

Specification that connect partially or fully composite beams to steel columns with flexural resistance provided by a force couple achieved with steel reinforcement in the slab and a steel seat angle or similar connection at the bottom flange.

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

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

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

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

1. SCOPE

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

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

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

2. REFERENCED CODES AND STANDARDS

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

American Concrete Institute
ACI 318-95[1]

American Iron and Steel Institute
Specification for the Design of Cold-Formed Steel Structural Members, 1996 Edition[2]

American Society of Civil Engineers
ASCE 3-91

3. SEISMIC DESIGN CATEGORIES

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

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

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

4. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

The loads and load combinations shall be those in Part I Section 4, including the requirements for the amplified horizontal earthquake load $\Omega_o Q_E$. The System Overstrength Factor Ω_o shall be as defined in the Applicable Building Code. In the absence of such definition, Ω_o shall be as listed in Table II-4-1.

TABLE II-4-1
System Overstrength Factor, Ω_o

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

5. MATERIALS

5.1. Structural Steel

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

5.2. Concrete and Steel Reinforcement

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

1. The specified minimum compressive strength of concrete in composite members shall equal or exceed 2.5 ksi.

2. For the purposes of determining the nominal strength of composite members, f'_c shall not be taken as greater than 10 ksi for normal-weight concrete nor 4 ksi for lightweight concrete.

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

6. COMPOSITE MEMBERS

6.1. Scope

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

6.2. Composite Floor and Roof Slabs

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

6.2a. Details shall be designed to transfer forces between the diaphragm and Boundary Members, Collector Elements, and elements of the horizontal framing system.

6.2b. The nominal shear strength of composite diaphragms and concrete-filled steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with ACI 318 excluding Chapter 22. Alternatively, the composite diaphragm design shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

6.3. Composite Beams

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

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

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

where
Y_{con} = distance from the top of the steel beam to the top of concrete, in.
d_b = depth of the steel beam, in.
F_y = specified minimum yield strength of the steel beam, ksi.
E_s = elastic modulus of the steel beam, ksi.

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

6.4. Reinforced-concrete-encased Composite Columns

This Section is applicable to columns that: (1) consist of reinforced-concrete-encased structural steel sections with a structural steel area that comprises at least 4 percent of the total composite-column cross-section; and (2) meet the additional limitations in LRFD Specification Section I2.1. Such columns shall meet the requirements in LRFD Specification Chapter I, except as modified in this Section. Additional requirements, as specified for intermediate and special seismic systems in Sections 6.4b and 6.4c, shall apply as required in the descriptions of the composite seismic systems in Sections 8 through 17.

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

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

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

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

6.4a. Ordinary Seismic System Requirements

The following requirements for reinforced-concrete-encased composite columns are applicable to all composite systems:

1. The nominal shear strength of the column shall be determined as the nominal shear strength of the structural shape plus the nominal shear strength that is provided by the tie reinforcement in the reinforced-concrete encasement. The nominal shear strength of the structural steel section shall be determined in accordance with LRFD Specification Section F2. The nominal shear strength of the tie reinforcement shall be determined in accordance with ACI 318 Sections 11.5.6.2 through 11.5.6.8. In ACI 318 Sections 11.5.6.4 and 11.5.6.8, the dimension b_w shall equal the width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear. The nominal shear strength shall be multiplied by ϕ_v equal to 0.75 to determine the design shear strength.

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

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

   b. If an external member is framed directly to the reinforced concrete to transfer a vertical reaction V_u, shear connectors shall be provided to transfer the force $V_uA_sF_y/P_n$ between the structural steel section and the reinforced concrete, where A_s, F_y and P_n are as defined above.

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

3. The maximum spacing of transverse ties shall be the least of the following requirements:

   a. one-half the least dimension of the section,

   b. 16 longitudinal bar diameters

c. 48 tie diameters

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

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

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

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

6.4b. Intermediate System Requirements

Reinforced-concrete-encased composite columns in intermediate seismic systems shall meet the following requirements in addition to those in Section 6.4a:

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

a. one-half the least dimension of the section

b. 8 longitudinal bar diameters

c. 24 tie bar diameters

d. 12 in.

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

a. one-sixth the vertical clear height of the column

b. The maximum cross-sectional dimension

c. 18 in.

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

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

6.4c. Special Seismic System Requirements

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

1. The required axial strength for reinforced-concrete-encased composite columns and splice details shall meet the requirements in Part I Section 8.

2. Longitudinal load-carrying reinforcement shall meet the requirements in ACI 318 Section 21.4.3.

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

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

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

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

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

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

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

4. Reinforced-concrete-encased composite columns in braced frames with axial compression forces that are larger than 0.2 times P_o shall have transverse reinforcement as specified in Section 6.4c3.c over the total element length. This requirement need not be satisfied if the nominal strength of the reinforced-concrete-encased steel section alone is greater than $1.0D+0.5L$.

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

6. Reinforced-concrete-encased composite columns that are used in C-SMF shall meet the following requirements:

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

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

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

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

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

6.5. Concrete-filled Composite Columns

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

6.5a. The design shear strength of the composite column shall be the design shear strength of the structural steel section alone.

6.5b. In addition to the requirements in Section 6.5a, in the special seismic systems described in Sections 9, 13 and 14, the design forces and column splices for concrete-filled composite columns shall also meet the requirements in Part I Section 8.

6.5c. Concrete-filled composite columns used in C-SMF shall meet the following requirements in addition to those in Sections 6.5a. and 6.5b:

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

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

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

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

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

7. COMPOSITE CONNECTIONS

7.1. Scope

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

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

7.2. General Requirements

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

7.3. Nominal Strength of Connections

The nominal strength of connections in composite structural systems shall be determined on the basis of rational models that satisfy both equilibrium of internal forces and the strength limitation of component materials and elements based upon potential limit states. Unless the connection strength is determined by analysis and testing, the models used for analysis of connections shall meet the requirements in Sections 7.3a through 7.3d.

7.3a. When required, force shall be transferred between structural steel and reinforced concrete through direct bearing of headed shear studs or suitable alternative devices, by other mechanical means, by shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer, or by a combination of these means. Any potential bond strength between structural steel and reinforced concrete shall be ignored for the purpose of the connection force transfer mechanism.

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

7.3b. The required strength of structural steel components in composite connections shall not exceed the design strengths as determined in Part I and the LRFD Specification. Structural steel elements that are encased in confined reinforced concrete are permitted to be considered to be braced against out-of-plane buckling. Face Bearing Plates consisting of stiffeners between the flanges of steel beams are required when beams are embedded in reinforced concrete columns or walls.

7.3c. The nominal shear strength of reinforced-concrete-encased steel panel-zones in beam-to-column connections shall be calculated as the sum of the nominal strengths of the structural steel and confined reinforced concrete shear elements as determined in Part I Section 9.3 and ACI 318 Section 21.5, respectively. The strength reduction (resistance) factors for reinforced concrete shall be as given in ACI 318 Appendix C.

7.3d. Reinforcement shall be provided to resist all tensile forces in reinforced concrete components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as appropriate, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with ACI 318 Chapter 12. Additionally, development lengths for the systems described in Sections 9, 13, 14, 16 and 17 shall meet the requirements in ACI 318 Section 21.5.4. Connections shall meet the following additional requirements:

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

2. For connections between structural steel or composite beams and reinforced concrete or reinforced-concrete-encased composite columns, transverse hoop reinforcement shall be provided in the connection region to meet the requirements in ACI 318 Section 21.5, except for the following modifications:

   a. Structural steel sections framing into the connections are considered to provide confinement over a width equal to that of face bearing stiffener plates welded to the beams between the flanges.
   b. Lap splices are permitted for perimeter ties when confinement of the splice is provided by Face Bearing Plates or other means that prevents spalling of the

concrete cover in the systems described in Sections 10, 11, 12 and 15.

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

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

8.1 Scope

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

8.2. Columns

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

8.3. Composite Beams

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

8.4. Partially Restrained (PR) Moment Connections

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

9. COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

9.1. Scope

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

9.2. Columns

Composite columns shall meet the requirements for special seismic systems in Sections 6.4 or 6.5. Reinforced concrete columns shall meet the requirements in ACI 318 Chapter 21, excluding Section 21.8.

9.3. Beams

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

9.4. Moment Connections

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

9.5. Column-Beam Moment Ratio

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

a. The flexural strength of the composite column M^*_{pc} shall meet the requirements in LRFD Specification Chapter I with consideration of the applied axial load, P_u.

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

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

10. COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)

10.1. Scope

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

10.2. Columns

Composite columns shall meet the requirements for intermediate seismic systems in Section 6.4 or 6.5. Reinforced concrete columns shall meet the requirements in ACI 318 Section 21.8.

10.3. Beam

Structural steel and composite beams shall meet the requirements in the LRFD Specification.

10.4. Moment Connections

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

1. The connection design strength shall meet or exceed the forces associated with plastic hinging of the beams adjacent to the connection.

2. The connection design strength shall meet or exceed the required strength generated by Load Combinations 4-1 or 4-2 in Part I.

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

11. COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)

11.1 Scope

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

11.2. Columns

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

11.3. Beams

Structural steel and composite beams shall meet the requirements in the LRFD Specification.

11.4. Moment Connections

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

12. COMPOSITE ORDINARY BRACED FRAMES (C-OBF)

12.1. Scope

This Section is applicable to concentrically and eccentrically braced frame systems that consist of either composite or reinforced concrete columns, structural steel or composite beams, and structural steel or composite braces. C-OBF shall be designed assuming that under the Design Earthquake limited inelastic action will occur in the beams, columns, braces, and/or connections.

12.2. Columns

Reinforced-concrete-encased composite columns shall meet the requirements for ordinary seismic systems in Sections 6.4. Concrete-filled composite columns shall meet the requirements in Section 6.5. Reinforced concrete columns shall meet the requirements in ACI 318 excluding Chapter 21.

12.3. Beams

Structural steel and composite beams shall meet the requirements in the LRFD Specification.

12.4. Braces

Structural steel braces shall meet the requirements in the LRFD Specification. Composite braces shall meet the requirements for composite columns in Section 12.2.

12.5. Connections

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

13. COMPOSITE CONCENTRICALLY BRACED FRAMES (C-CBF)

13.1. Scope

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

13.2. Columns

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

13.3. Beams

Structural steel and composite beams shall meet the requirements in the LRFD Specification.

13.4. Braces

Structural steel braces shall meet the requirements for OCBF in Part I Section 14. Composite braces shall meet the requirements for composite columns in Section 13.2.

13.5. Bracing Connections

Bracing connections shall meet the requirements in Section 7 and Part I Section 14.

14. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

14.1. Scope

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

14.2. Columns

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

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

14.3. Links

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

14.4. Braces

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

14.5. Connections

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

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

15.1. Scope

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

15.2. Boundary Members

15.2a. When unencased structural steel sections function as Boundary Members in reinforced concrete infill panels, the structural steel sections shall meet the requirements in the LRFD Specification. The required axial strength of the Boundary Member shall be determined assuming that the shear forces are carried by the reinforced concrete wall and the entire gravity and overturning forces are carried by the Boundary Members in conjunction with the shear wall. The reinforced concrete wall shall meet the requirements in ACI 318 excluding Chapter 21.

15.2b. When fully reinforced-concrete-encased structural steel sections function as Boundary Members in reinforced concrete infill panels, the analysis shall be based upon a transformed concrete section using elastic material properties. The wall shall meet the requirements in ACI 318 excluding Chapter 21. When the reinforced-concrete-encased structural steel Boundary Member qualifies as a composite column as defined in LRFD Specification Chapter I, it shall be designed to meet the ordinary seismic system requirements in Section 6.4. Otherwise, it shall be designed as a composite column to meet the requirements in ACI 318.

15.2c. Headed shear studs or welded reinforcement anchors shall be provided to transfer vertical shear forces between the structural steel and reinforced concrete. Headed shear studs, if used, shall meet the requirements in LRFD Specification Chapter I. Welded reinforcement anchors, if used, shall meet the requirements in AWS D1.4.

15.3. Coupling Beams

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

15.3a. Coupling Beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the maximum possible combination of moment and shear that can be generated by the nominal bending and shear strength of the Coupling Beam. The embedment length shall be considered to begin inside the first layer of confining reinforcement in the wall Boundary Member. Connection strength for the transfer of loads between the Coupling Beam and the wall shall meet the requirements in Section 7.

15.3b. Vertical wall reinforcement with design axial strength equal to the nominal shear strength of the Coupling Beam shall be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement shall extend a distance of at least one tension development length above and below the flanges of the Coupling Beam. It is permitted to use vertical reinforcement placed for other purposes, such as for vertical Boundary Members, as part of the required vertical reinforcement.

16. SPECIAL REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL

STEEL ELEMENTS (C-SRCW)

16.1. Scope

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

16.2. Boundary Members

16.2a. In addition to the requirements in Section 15.2a, unencased structural steel columns shall meet the requirements in Part I Sections 5, 6 and 8.

16.2b. In addition to the requirements in Section 15.2b, the requirements in this Section shall apply to walls with reinforced-concrete-encased structural steel Boundary Members. The wall shall meet the requirements in ACI 318 including Chapter 21. Reinforced-concrete-encased structural steel Boundary Members that qualify as composite columns in LRFD Specification Chapter I shall meet the special seismic system requirements in Section 6.4. Otherwise, such members shall be designed as composite compression members to meet the requirements in ACI 318 including the special seismic requirements for Boundary Members in Chapter 21. Transverse reinforcement for confinement of the composite Boundary Member shall extend a distance of $2h$ into the wall where h is the overall depth of the Boundary Member in the plane of the wall.

16.2c. Headed shear studs or welded reinforcing bar anchors shall be provided as specified in Section 15.2c. For connection to unencased structural steel sections, the nominal strength of welded reinforcing bar anchors shall be reduced by 25 percent from their Static Yield Strength.

16.3 Coupling Beams

16.3a. In addition to the requirements in Section 15.3a, structural steel Coupling Beams shall meet the requirements in Part I Sections 15.2a through 15.2f, 15.3b and 15.3c. When required in Part I Section 15.3b, the coupling rotation shall be assumed as 0.08 radians unless a smaller value is justified by rational analysis of the inelastic deformations that are expected under the Design Earthquake. Face Bearing Plates shall be provided on both sides of the Coupling Beams at the face of the reinforced concrete wall. These stiffeners shall meet the detailing requirements in Part I Section 15.3a.

16.3b. Vertical wall reinforcement as specified in Section 15.3b shall be confined by transverse reinforcement that meets the requirements for Boundary Members in ACI 318 Section 21.2.6.

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

17.1. Scope

This Section is applicable to structural walls consisting of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite Boundary Members.

17.2 Wall Element

17.2a. Nominal Shear Strength

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

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

where

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

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

17.2b. The steel plate shall be adequately stiffened by encasement or attachment to the reinforced concrete if it can be demonstrated with an elastic plate buckling analysis that the composite wall can resist a nominal shear force equal to V_{ns}. The concrete thickness shall be a minimum of 4 in. on each side when concrete is provided on both sides of the steel plate and 8 in. when concrete is provided on one side of the steel plate. Headed shear stud connectors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet the detailing requirements in ACI 318 Section 14.3. The reinforcement ratio in both directions shall not be less than 0.0025; the maximum spacing between bars shall not exceed 18 in.

17.2c. The steel plate shall be continuously connected on all edges to structural steel framing and Boundary Members with welds and/or slip-critical high-strength bolts to develop the nominal shear strength of the plate. The design strength of welded and bolted connectors shall meet the additional requirements in Part I Section 7.

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

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

Part III
Allowable Stress Design (ASD) Alternative

As an alternative to the Load and Resistance Factor Design (LRFD) provisions for structural steel design in Part I, the use of the Allowable Stress Design (ASD) provisions in this Part is permitted. All requirements of Part I shall be met except as modified or supplemented in this Part. When using this Part, the terms "LRFD Specification", "FR" and "PR" in Part I shall be taken as "ASD Specification" (AISC, 1989), "Type 1" and "Type 3", respectively.

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

1. SCOPE

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

These Provisions shall be applied in conjunction with the AISC *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design*, hereinafter referred to as the ASD Specification. All members and connections in the Seismic Force Resisting System shall be proportioned as required in the ASD Specification to resist the applicable load combinations and shall meet the requirements in these Provisions.

Part III includes the Part I Glossary and Appendix S.

2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

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

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

American Institute of Steel Construction
S*pecification for Structural Steel Buildings—Allowable Stress Design and Plastic Design*, June 1, 1989.

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

Research Council on Structural Connections
Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts, November 13, 1985, reaffirmed with modification to Appendix A only, June 3, 1994.

Substitute the following for Part I Section 4 in its entirety:

4. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

4.1. Loads and Load Combinations

In addition to loads and load combinations involving non-seismic cases specified by the Applicable Building Code, the following seismic Load Combinations shall be investigated, except as modified throughout these Provisions.

$$1.2D \pm 1.0E + 0.5L + 0.2S \quad \text{(4-a)}$$

$$0.9D \pm (1.3W \text{ or } 1.0E) \quad \text{(4-b)}$$

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

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

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

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

Exception: The load factor on L in load combination 4-a and 4-1 shall equal 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.

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

TABLE I-4-1
System Overstrength Factor, Ω_o

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

4.2. Nominal Strengths

The nominal strengths of members and connections shall be determined as follows:

4.2a. Replace ASD Specification Section A5.2 to read: “The nominal strength of structural steel members and connections for resisting seismic forces acting alone or in combination with dead and live loads shall be determined by multiplying 1.7 times the allowable stresses in Section D, E, F, G, H, J, and K. The 1/3 allowable stress increase shall not be applied in conjunction with this factor.”

4.2b. Amend the first paragraph of ASD Specification Section N1 by deleting “or earthquake” and adding: “The nominal strength of members and connections shall be determined by the requirements contained herein. Except as modified in these provisions, all pertinent requirements of Chapters A through M shall govern."

4.2c. In ASD Specification Section H1 the definition of F_e' shall read as follows:

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

where:

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

4.3. Design Strengths

4.3a. The design strengths of structural steel members and connections subjected to seismic forces in combination with other prescribed loads shall be determined by converting allowable stresses into nominal strengths and multiplying such nominal strengths by the resistance factors herein.

4.3b. Resistance factors ϕ for use in Part III shall be as follows:

Tension	
yielding	0.9
rupture	0.75
Compression	
buckling	0.85
Flexure	
yielding	0.9
rupture	0.75
Shear	
yielding	0.9
rupture	0.75
Torsion	
yielding	0.9
buckling	0.9
CJP groove welds	
tension or compression normal to effective area	0.9 for base metal 0.9 for weld metal
shear on effective area	0.9 for base metal 0.8 for weld metal
PJP groove welds	
compression normal to effective area	0.9 for base metal 0.9 for weld metal
tension normal to effective area	0.9 for base metal 0.8 for weld metal
shear parallel to axis of weld	0.75 for weld metal
Fillet welds	
shear on effective area	0.75 for weld metal
Plug or slot welds	
shear parallel to faying surface	

(on effective area)	0.75 for weld metal
Bolts	
tension rupture, shear rupture, combined tension and shear	0.75
slip resistance for bolts in standard holes, oversized holes, and short-slotted holes	1.0
slip resistance for bolts in long-slotted holes with the slot perpendicular to the direction of the slot	1.0
slip resistance for bolts in long-slotted holes with the slot parallel to the direction of the slot	0.85
Connecting elements	
tension yielding, shear yielding	0.9
bearing strength at bolt holes, tension rupture, shear rupture, block shear rupture	0.75
contact bearing	0.75 for bearing on steel 0.6 for bearing on concrete
Flanges and webs with concentrated forces	
local flange bending, compression buckling of web	0.9
local web yielding	1.0
web crippling, panel-zone web shear	0.75
sidesway web buckling	0.85

7. CONNECTIONS, JOINTS AND FASTENERS

7.2. Bolted Joints

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

7.2d. The design resistance to shear and combined tension and shear of bolted joints shall be determined in accordance with the ASD Specification Sections J3.5 and J3.7, except that the allowable bearing stress at bolt holes F_p shall not be taken greater than $1.2F_u$.

8. COLUMNS

Substitute the following for the first paragraph of Part I Section 8.3 in its entirety:

8.3. Column Splices

The design strength of column splices shall exceed the required strength determined from Section 8.2 and from Load Combinations 4-1 and 4-2.

9. SPECIAL MOMENT FRAMES

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

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

When $P_u \leq 0.75P_y$,

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

When $P_u > 0.75P_y$,

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

where

t_p = total thickness of panel-zone including doubler plate(s), in.

d_c = overall column depth, in.

b_{cf} = width of the column flange, in.

t_{cf} = thickness of the column flange, in.

d_b = overall beam depth, in.

F_y = specified minimum yield strength of the panel-zone steel, ksi.

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

9.7.b.1 The required column strength shall be determined from Load Combination 4-b, except that E shall be taken as the lesser of:

a. The amplified earthquake force $\Omega_o Q_E$.

b. 125 percent of the frame design strength based upon either the beam design flexural strength or panel-zone design shear strength.

12. SPECIAL TRUSS MOMENT FRAMES

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

12.4 Nominal Strength of Non-special Segment Members

All members and connections of STMF, except those in the special segment in Section 12.2., shall have a design strength to resist Load Combinations 4-a and 4-b and the lateral loads necessary to develop the expected vertical nominal shear strength in all segments V_{ne} given as: [balance to remain unchanged]

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

12.6 Lateral Bracing

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

13. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

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

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

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

3. A beam that is intersected by braces shall be designed to resist the effects of Load Combinations 4-a and 4-b except that a load Q_b shall be substituted for the term E. Q_b is the maximum unbalanced vertical load effect applied to the beam by the braces. This load effect shall be calculated using a minimum of P_y for the brace in tension and a maximum of 0.3 times $\phi_c P_n$ for the brace in compression.

14. ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

Substitute the following for Part I Section 14.4a.1 in its entirety:

1. The design strength of brace members shall be at least 1.5 times the required strength using Load Combinations 4-a and 4-b.

Substitute the following for Part I Section 14.4a.3. in its entirety:

3. A beam that is intersected by braces shall be designed to support the effects of all tributary dead and live loads as required by the ASD Specification assuming that the bracing is not present.

Commentary

April 15, 1997

Part I—Structural Steel Buildings

Experience from the 1994 Northridge and 1995 Kobe earthquakes significantly expanded the known response characteristics of structural steel building systems, particularly welded steel moment frames. Shortly after the Northridge earthquake, the SAC Joint Venture[1] initiated a comprehensive study of the seismic performance of steel moment frames. Funded by the Federal Emergency Management Agency (FEMA), SAC is developing guidelines for structural engineers, building officials and other interested parties for the evaluation, repair, modification and design of welded steel moment frame structures in seismic regions. AISC is an active participant in SAC activities.

Many recommendations in the SAC *Interim Guideline* (FEMA, 1995) form the basis of new provisions herein. In addition, a number of other relevant research reports have been referenced. While research is ongoing, this revision of the AISC Seismic Provisions represents the best available knowledge to date. These Provisions were developed simultaneously and cooperatively with the revisions that the Building Seismic Safety Council (BSSC) will provide for the 1997 NEHRP Provisions (FEMA, 1997a). Accordingly, it is anticipated that this document will form the basis for steel seismic design provisions in the 1997 NEHRP Provisions as well as those in the 2000 International Building Code (IBC), which is currently under development by the International Code Council (ICC).

C1. SCOPE

Structural steel building systems in seismic regions are generally expected to dissipate seismic input energy through controlled inelastic deformations of the structure. These Provisions supplement the AISC LRFD Specification (AISC, 1993) for such applications. The seismic design forces that are specified in the building codes have been set with consideration of the energy dissipation generated during inelastic response.

C2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The specifications, codes and standards referenced in Part I are listed with the appropriate revision date that was used in the development of Part I. While most of these documents are also referenced in the LRFD Specification, some have been revised since its publication in 1993.

C3. SEISMIC DESIGN CATEGORIES

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[1]A joint venture of the Structural Engineers Association of California (SEAOC), Applied Technology (ATC), and California Universities for Research in Earthquake Engineering (CUREe).

In order to design buildings to resist earthquake motions, each building is categorized depending upon its occupancy and use to establish the potential earthquake hazard that it represents. The determination of the required strength for use in design differs significantly in each specification or building code. The primary purpose of these Provisions is to provide the information necessary to determine the design strength of steel buildings. The following discussion provides a basic overview of the approach to categorization of building structures that is taken in several of the seismic codes or specifications, as well as the corresponding determination of the required strength and stiffness. For the variables required to assign Seismic Design Categories, limitations of height, vertical and horizontal irregularities, site characteristics, etc., the Applicable Building Code should be consulted.

In the 1997 NEHRP Provisions (FEMA, 1997a), buildings are assigned to one of three Seismic Use Groups, depending upon occupancy or use. Group III includes essential facilities, while Groups II and I include facilities with a lesser associated degree of public hazard. Buildings are then assigned to a Seismic Design Category based upon the Seismic Use Group, the seismicity of the site and the period of the building. Seismic Design Categories A, B and C are generally applicable to buildings in areas of low to moderate seismicity and special seismic provisions like those in these Provisions are not mandatory. However, seismic provisions are mandatory in Seismic Design Categories D, E and F, including consideration of system redundancy. Seismic Design Category D is generally applicable to buildings in areas of high seismicity and Seismic Use Group III buildings in areas of moderate seismicity. Seismic Design Categories E and F are generally applicable to buildings in Seismic Use Groups I and II and Seismic Use Group III, respectively, in areas of especially high seismicity.

In ASCE 7 (ASCE, 1995), buildings are assigned to one of four Occupancy Groups. Group IV, for example, includes essential facilities. Buildings are then assigned to a Seismic Performance Category based upon the Occupancy Group and the seismicity of the site. Seismic Design Categories A, B and C are generally applicable to buildings in areas of low to moderate seismicity and special seismic provisions like those in these Provisions are not mandatory. However, seismic provisions are mandatory in Seismic Design Categories D and E, which cover areas of high seismicity.

In the 1997 Uniform Building Code (ICBO, 1997a) and the 1996 SEAOC Seismic Provisions Appendix C (SEAOC, 1996), buildings are assigned to Seismic Design Categories based upon the Seismic Zone, Importance Factor and Soil Profile Type.

C4. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTH

The load factors and load combinations given herein and in LRFD Specification Section A4.1 are consistent with those given in ASCE 7 (ASCE, 1995), the 1997 NEHRP Provisions (FEMA, 1997a) and the 1997 Uniform Building Code (ICBO, 1997a). It is also anticipated that they will be consistent with those in the 2000 International Building Code, which is currently under development. The most notable modification from load factors and load combinations in some earlier editions of these Provisions is the reduction of the load factor on E to 1.0, which is consistent with the limit-states load model used in the current load specifications. For the design of structures subjected to impact loads, see LRFD Specification Section A4.2.

The earthquake load E in ASCE 7, the 1997 NEHRP Provisions and the 1997 Uniform Building Code is the combination of the horizontal seismic load effect and a simulated effect due to the vertical accelerations that would accompany the horizontal earthquake effects.

The load factors and load combinations account for the likelihood that, when several transient loads act in combination with the dead load, such as in the load case for combined dead, live and earthquake loads, two or more transient loads will not each be at their maximum lifetime values concurrently. While one transient load is at its maximum lifetime value, other transient loads are taken at their arbitrary-point-in-time value, which is the magnitude of that particular load that can be expected to act on the structure at any time. The most critical combined load effect may occur when one or more loads are not acting.

An amplification factor Ω_o to the horizontal earthquake load Q_E is prescribed for limited use in Load Combinations 4-1 and 4-2, primarily to account for the overstrength that is inherent in the type of system to be used when determining the required strength of connections.

The general relationship between the different structural steel systems is illustrated in Table I-C4-1 based upon similar information in the 1997 NEHRP Provisions. *R* is a seismic force reduction factor that is used to estimate the inherent overstrength and ductility of the Seismic Force Resisting System. C_d is an amplification factor that is used with the forces for strength design to calculate the seismic drift. The use of these factors should be consistent with that specified in the Applicable Building Code with due consideration of the limitations and modifications that are necessary therein due to such issues as building category, building height, vertical or horizontal irregularities, and site characteristics.

TABLE I-C4-1
DESIGN FACTORS FOR STRUCTURAL STEEL SYSTEMS

BASIC STRUCTURAL SYSTEM AND SEISMIC FORCE RESISTING SYSTEM	*R*	C_d
Systems designed and detailed to meet the requirements in the LRFD Specification but not the requirements of Part I	3	3
Systems designed and detailed to meet the requirements of both the LRFD Specification and Part I:		
Braced Frame Systems:		
Special Concentrically Braced Frames (SCBF)	6	5
Ordinary Concentrically Braced Frames (OCBF)	5	4½
Eccentrically Braced Frames (EBF)		
with moment connections at columns away from Link	8	4
without moment connections at columns away from Link	7	4
Moment Frame Systems:		
Special Moment Frames (SMF)	8	5½
Intermediate Moment Frames (IMF)	6	5
Ordinary Moment Frames (OMF)	4	3½
Special Truss Moment Frames (STMF)	7	5½
Dual Systems with SMF capable of resisting 25 percent of V:		
Special Concentrically Braced Frames (SCBF)	8	6½
Ordinary Concentrically Braced Frames (OCBF)	6	5
Eccentrically Braced Frames (EBF)		
with moment connections at columns away from Link	8	4
without moment connections at columns away from Link	7	4
Dual Systems with IMF * capable of resisting 25 percent of *V*:		
Special Concentrically Braced Frames (SCBF)	6	5
Ordinary Concentrically Braced Frames (OCBF)	5	4½
*OMF is permitted in lieu of IMF in Seismic Design Categories A, B and C.		

C5. STORY DRIFT

Story drift limits, like deflection limits, are commonly used in design to assure the serviceability of the structure, although they are variable because they depend upon the structural usage and contents. Such serviceability limit states are regarded as a matter of engineering judgment rather than absolute design limits (Fisher and West, 1990) and no specific design requirements are given in the LRFD Specification or these Provisions.

Research has shown that story drift limits, although primarily related to serviceability, also improve frame stability (P-Δ) and seismic performance because of the resulting additional strength and stiffness. Although some building codes, load standards and resource documents contain specific seismic drift limits, there are major differences among them as to what limit is specified and how the limit is applied. Furthermore, it is difficult to estimate the actual story drift in many cases, such as in moment frames that exhibit shear yielding of the panel-zones. Nevertheless, drift control is important to both the serviceability and the stability of the structure. As a minimum, the designer should use the drift limits specified in the Applicable Building Code.

The story drift limits in ASCE 7 (ASCE, 1995) and the 1997 NEHRP Provisions (FEMA, 1997a) are for comparison to an amplified story drift that approximates the difference in deflection between the top and bottom of the story under consideration during a large earthquake. The amplified story drift is determined by multiplying the horizontal component of the earthquake force E by a deflection amplification factor C_d, which is dependent upon the type of building system used; see Table I-C4-1.

C6. MATERIALS

C6.1. Material Specifications

The structural steels that are explicitly permitted for use in seismic design have been selected based upon their inelastic properties and weldability. In general, they meet the following characteristics: (1) a ratio of yield strength to tensile strength not greater than 0.85; (2) a pronounced stress-strain plateau at the yield strength; (3) a large inelastic strain capability (for example, tensile elongation of 20 percent or greater in a 2-in. gage length); and (4) good weldability. Other steels should not be used without evidence that the above criteria are met.

In this revision, ASTM A53 and ASTM A913 Grades 50 and 65 have been included in the list of explicitly permitted structural steels. ASTM A53 steel pipe is often used for bracing members in braced frames and meets the above criteria. ASTM A913 has been accepted for seismic applications by the AISC Committee on Specifications and by the ICBO Lateral Forces Committee. ASTM A913 Grade 65 is intended primarily for use in columns, especially in moment frames where a strong-column/weak-beam (SC/WB) concept is employed; see Commentary Section C9.6.

C6.2. Material Properties for Determination of Required Strength for Connections or Related Members

Brittle fracture of beam-to-column moment connections in the Northridge Earthquake resulted from a complex combination of variables. One of the many contributing factors was the failure to recognize that actual beam yield stresses are generally higher than the specified minimum yield stress F_y, which elevates the connection demand. In 1994, the Structural Shape Producers Council (SSPC) conducted a survey to determine the characteristics of current structural steel production (SSPC, 1994). FEMA (1995) recommended that the mean values of F_y from the SSPC study be used in calculations of demand on moment connections. It has been recognized subsequently that the same overstrength concerns also apply to other systems as well as to moment frames.

R_y is the ratio of expected yield strength F_{ye} to specified minimum yield strength F_y. It is used as a multiplier on the specified minimum yield strength when calculating the required strength of connections and other members that must withstand the development of inelasticity in another member. The specified values of R_y are somewhat lower than those that can be calculated using the mean values reported in the SSPC survey. Those values were skewed somewhat by the inclusion of a large number of smaller members, which typically have higher measured yield strengths than the larger members common in seismic design. The given values are considered to be reasonable averages, although it is recognized that they are not maxima. Alternatively, the expected yield strength F_{ye} can be determined by testing conducted in accordance with the requirements for the specified grade of steel. Refer to ASTM A370.

The higher values of R_y for ASTM A36 ($R_y = 1.5$) and ASTM A572 Grade 42 ($R_y = 1.3$) W-shapes are indicative of currently observed properties of these grades of steel. If the material being used in design was produced several years ago, it may be possible to use a reduced value of R_y based upon testing of the steel to be used or other supporting data (Galambos and Ravindra, 1978).

C6.3. Notch Tough Steel

The LRFD Specification requirements for notch toughness cover Groups 4 and 5 shapes and plate elements with thickness that is greater than or equal to 2 in. in tension applications. In these Provisions, this requirement is extended to cover: (1) all Group 4 and 5 shapes that are part of the Seismic Force Resisting System; (2) ASTM Group 3 shapes that are part of the Seismic Force Resisting System with flange thickness greater than or equal to 1½ in.; and, (3) plate elements with thickness greater than or equal to 1½ in. that are part of the Seismic Force Resisting System, such as the flanges of built-up girders. Because other shapes and plates are generally subjected to enough cross-sectional reduction during the rolling process that the resulting notch toughness will exceed that required above (Cattan, 1995), specific requirements have not been included herein.

For rotary-straightened W-shapes, an area of reduced notch toughness has been documented in a limited region of the web immediately adjacent to the flange as illustrated in Figure C-6.1. Preliminary recommendations have been issued (AISC, 1997) and AISC is currently exploring the associated implications for design and construction. It is anticipated that recommendations will be forthcoming, albeit after the publication of this document. For this reason, the reader is encouraged to maintain an awareness of AISC recommendations as they become available.

C7. CONNECTIONS, JOINTS AND FASTENERS

C7.2. Bolted Joints

The potential for full reversal of design load and likelihood of inelastic deformations of members and/or connected parts necessitates that fully tensioned bolts be used in bolted joints in the Seismic Force Resisting System. However, earthquake motions are such that slip cannot be prevented in all cases, even with slip-critical connections. Accordingly, these Provisions call for bolted joints to be proportioned as fully tensioned bearing joints but with faying surfaces prepared as for Class A or better slip-critical connections. That is, bolted connections can be proportioned with design strengths for bearing connections as long as the faying surfaces are still prepared to provide a minimum slip coefficient $\mu = 0.33$. The resulting nominal amount of slip resistance will minimize damage in more moderate seismic events. Additionally, the sharing of design load between welds and bolts on the same faying surface is not permitted.

To prevent excessive deformations of bolted joints due to slip between the connected plies under earthquake motions, the use of holes in bolted joints in the Seismic Force Resisting System is limited to standard holes and short-slotted holes with the direction of the slot perpendicular to the line of force. An exception is provided for alternative hole types that are justified as a part of a tested assembly.

To prevent excessive deformations of bolted joints due to bearing on the connected material, the bearing strength is limited by the deformation-considered option in LRFD Specification Section J3.10 ($\phi R_n = 0.75 \text{x} 2.4 dtF_u$). The philosophical intent of this limitation in the LRFD Specification is to limit the bearing deformation to an approximate maximum of ¼ in. It should be recognized, however, that the actual bearing force in a seismic event may be much larger than that anticipated in design and the actual deformation of holes may exceed this theoretical limit. Nonetheless, this limit will effectively minimize damage in moderate seismic events.

Tension or shear fracture, bolt shear, and block shear rupture are examples of limit states that generally result in non-ductile failure of connections. As such, these limit states are undesirable as the controlling limit state for connections that are part of the Seismic Force Resisting System. Accordingly, it is required that connections be configured such that a ductile limit state in the member or connection, such as yielding or bearing deformation, controls the design strength.

C7.3. Welded Joints

The general requirements for welded joints are given in AWS D1.1 (AWS, 1996), wherein a Welding Procedure Specification (WPS) is required for all welds. Approval by the Engineer of Record of the WPS to be used is required in this Specification.

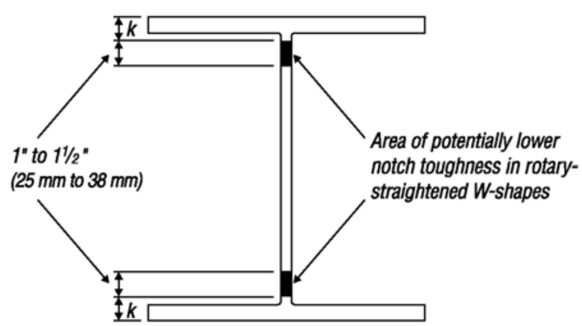


Fig. C-6.1. "k-area".

For CJP groove-welded joints in the Seismic Force Resisting System, weld metal notch toughness is required in these Provisions. Although the *SAC Interim Guideline* (FEMA, 1997b) indicates the acceptability of electrodes that provide a specified minimum toughness of 20 ft-lbs at 0 degrees F, electrodes with a specified minimum toughness of 20 ft-lbs at minus 20 degrees F have been utilized in most testing to date. For this reason, and to account for minor variations between manufacturer qualification testing and end-use results, a specified minimum toughness of 20 ft-lbs at minus 20 degrees F has been conservatively specified in these Provisions. Note that it is not the intent of these Provisions to require testing of either the welding procedure or production welds.

Many operations during fabrication, erection, and the subsequent work of other trades have the potential to create discontinuities in the Seismic Force Resisting System. When located in regions of potential inelasticity, such discontinuities are required to be repaired by the responsible subcontractor as required by the Engineer of Record. Discontinuities should also be repaired in other regions of the Seismic Force Resisting System when the presence of the discontinuity would otherwise be detrimental to its performance. The responsible subcontractor should propose a repair procedure for the approval of the Engineer of Record. Repair may be unnecessary for some discontinuities, subject to the approval of the Engineer of Record.

C8. COLUMNS

C8.2. Column Strength

The axial forces that are generated during earthquake motions in columns that are part of the Seismic Force Resisting System are expected to exceed those calculated using the code-specified seismic forces for several reasons, including: (1) the reduction in lateral force for use in analysis of an elastic model of the structure; (2) the underestimation of the overturning forces in the analysis; and (3) the concurrent occurrence of vertical accelerations that are not explicitly specified as a required load. The amplifications required in this Section represent an approximation of these actions and provide an upper bound for the required axial strength. Load Combinations 4-1 and 4-2 account for these effects with a minimum required compressive strength and a minimum required tensile strength, respectively, and are to be applied without consideration of any concurrent flexural loads on the column. The Ω_o term has been developed in conjunction with the 1997 NEHRP Provisions (FEMA, 1997a) to account for these effects in a simplified form.

The exceptions provided in Section 8.2c represent self-limiting conditions wherein the required axial strength need not exceed the capability of the structural system to transmit axial loads to the column. For example, because a spread footing foundation can only provide a certain resistance to uplift, there is a limit to the force that the system can transmit to a column. Conversely, the uplift resistance of a pile foundation that is designed primarily for compressive forces may significantly exceed the required tensile strength for the column. If so, this would not represent a system strength limit.

C8.3. Column Splices

The design strength of a column splice is required to equal or exceed both the required strength determined in Section 8.2 and the required strength for axial, flexural and shear effects at the splice location determined from LRFD Specification Load Combinations A4-1 through A4-6.

Column splices are required to be located away from the beam-to-column connection to reduce the effects of flexure. For typical buildings, the 4-ft minimum distance requirement will control. When located 4 to 5 ft above the floor level, field erection and construction of the column splice will generally be simplified due to increased accessibility and convenience.

Partial-joint-penetration groove welded splices of thick column flanges exhibit virtually no ductility under tensile loading (Popov and Steven, 1977; Bruneau et al., 1987). In recognition of this behavior, a 100 percent increase in required strength is stipulated for column splices that are made with partial-joint-penetration groove welds.

The calculation of the minimum required strength in Section 8.3a.2, as revised, includes the overstrength factor R_y. This results in a minimum required strength that is not less than 50 percent of the expected axial yield strength of the column flanges.

The possible occurrence of tensile forces in column splices utilizing partial-joint-penetration groove welds during a maximum probable earthquake should be considered. When tensile forces are possible, it is suggested that some restraint be provided against relative lateral movement between the spliced column shafts. For example, this can be achieved with the use of flange splice plates. Alternatively, web splice plates that are wide enough to maintain the general alignment of the spliced columns can be used. Shake-table experiments have shown that, when columns that are unattached at the base reseat themselves after lifting, the performance of a steel frame remains tolerable (Huckelbridge and Clough, 1977).

These provisions are applicable to common frame configurations. Additional considerations may be necessary when flexure dominates over axial compression in columns in moment frames, and in end columns of tall narrow frames where overturning forces can be very significant. The designer should review the conditions found in columns in buildings with tall story heights, when large changes in column sizes occur at the splice, or when the possibility of column buckling in single curvature over multiple stories exists. In these and similar cases, special column splice requirements may be necessary for minimum design strength and/or detailing.

In the 1992 AISC Seismic Provisions, beveled transitions between elements of differing thickness and or width were not generally required for butt splices in columns subject to seismic forces. Although no column splices are known to have failed in the Northridge Earthquake or previous earthquakes, this provision is no longer considered to be prudent given the concern over stress concentrations, particularly at welds. Moment frame systems are included in this requirement because inelastic analyses commonly indicate that large moments can be expected at any point along the column length, despite the indications of elastic analysis that moments are low at the mid-height of columns in moment frames that are subjected to lateral loads. Column splices in braced frames can also be subject to tension due to overturning effects. Accordingly, bevelled transitions are required for all systems with CJP groove-welded column splices. An exception to the requirements for beveled transitions is provided when partial-joint-penetration groove welds are acceptable.

C9. SPECIAL MOMENT FRAMES (SMF)

General Comments for Commentary Sections C9, C10 and C11

These Provisions include three types of steel moment frames: Special Moment Frames (SMF) in Section 9, Intermediate Moment Frames (IMF) (new) in Section 10, and Ordinary Moment Frames (OMF) in Section 11. The provisions for these three moment-frame types have been written to recognize the lessons learned from the Northridge and Kobe Earthquakes, and from the subsequent research performed by the SAC Joint Venture for FEMA. The reader is referred to SAC (1995a through 1995g) and FEMA (1995, 1997a and 1997b) for an extensive discussion of these lessons and recommendations to mitigate the conditions observed. Commentary on specific provisions in Section C9 is based primarily on FEMA (1995) and FEMA (1997b).

The prescriptive moment-frame connection that was included in the 1992 AISC Seismic Provisions was primarily based upon testing that was conducted in the early 1970s (Popov and Stephen, 1972) and indicated that, for the sizes and material strengths tested, a moment connection with complete-joint-penetration groove welded flanges and a welded or bolted web connection could accommodate inelastic rotations in the range of 0.01 to 0.015 radians. It was judged by engineers at the time that such rotations, which corresponded to building drifts in the range of 2 to 2½ percent were sufficient for adequate frame performance. As a result of the investigations that have been conducted subsequently to the Northridge earthquake, it has been recognized that many changes took place in materials, welding, frame configurations and member sizes in the years succeeding those tests that make their results unsuitable as a basis for current designs. Additionally, recent analyses using time histories from certain near-fault earthquakes and including P-Δ effects demonstrate that drift demands significantly exceeding the previously assumed range are possible (Krawinkler and Gupta, 1998).

The three frame types included in these Provisions offer three different levels of expected seismic inelastic rotation capability. SMF, IMF and OMF are designed to accommodate 0.03, 0.02 and 0.01 radians, respectively. If one recognizes that the elastic drift of typical moment frames is usually in the range of 0.01 radians and that the inelastic rotation of the beams is approximately equal to the inelastic drift, it can be seen that these frames can accommodate total drifts in the range of 0.04, 0.03 and 0.02 radians, respectively. Additionally, it can be seen that even the inelastic rotation capability expected of the OMF in these Provisions may be higher than that which can be accommodated reliably by connections, the tests of which formed the basis of previous designs.; thus, the need for improved provisions for moment-frame connections.

Although it is common to visualize that the inelastic rotations in moment frames occur at beam or column “hinges”, analysis and testing provide clear evidence that the inelastic rotations consist of a combination of the flexural deformations at the hinges and shear deformations of the panel-zones, unless the column webs are unusually thick. The contribution of the panel-zone to inelastic rotation is considered to be beneficial, provided that it is limited to a magnitude that neither significantly kinks the column flanges at the beam-flange-to-column-flange welds nor leads to significant column damage. The amount of panel-zone deformation that a given connection will have and how much it will accommodate can only be determined by testing.

Based upon the recommendations in FEMA (1995) and FEMA (1997b), it is required in these Provisions that connections for all three types of moment frames be based upon testing. An exception wherein testing is not required is provided for OMF connections, which can be proportioned following a set of prescriptive design rules that have been demonstrated in testing to provide adequate performance. The intent in these Provisions is not to require specific tests for each design, except where the design is unique and there are no published or otherwise available tests that adequately represent the conditions being used. For many commonly employed combinations of beam and column sizes, there are readily available test reports in publications of AISC, FEMA, and others, including FEMA (1997c) and NIST/AISC (1998).

C9.1. Scope

Special Moment Frames (SMF) are intended to provide for significant inelastic deformations. As noted above, the intent is for the majority of the inelastic deformation to take place as rotation in beam "hinges", with some inelastic deformation permitted in the panel-zone of the column. As also noted previously, the connections for these frames are required to be based upon tests that demonstrate the capability of the connection to provide an inelastic rotation of at least 0.03 radians under conditions of the required loading protocol. The other provisions are intended to limit or prevent panel-zone distortion, column hinging and local buckling, any of which might lead to inadequate frame performance in spite of good connection performance.

C9.2. Beam-to-Column Joints and Connections

C9.2a. This section describes the requirements for the tested connections as noted above. Reference is made to Appendix S, which provides the requirements for testing that are applicable to tests performed specifically for the design being used, or to similar tests performed by others for which reports are available, and upon which the design is to be based.

As noted, extrapolation and interpolation are permitted when it can be shown that similar conditions exist. Specific guidance is provided in Appendix S on extrapolation and interpolation of member sizes, and is permitted to be based upon rational analysis. In any case, it is required to be demonstrated that each member, connection element, and joint in the connection will be subjected to conditions (e.g., stress distributions, distortions, residual stresses, etc.) that are similar to those of the tested connections that are used as the basis of the design. Of course, the conditions and quality of the actual construction of the connections is required to be similar to that reported for the tests to achieve similar performance.

Limitations are placed on permissible differences between the tested yield strength and F_{ye} for beams, columns and connection elements. It is not intended that these limitations be applied retroactively to the existing database of qualification tests. Rather, these requirements are intended to apply for use in new qualification testing.

C9.2b. Acceptance criteria for connections that are qualified by testing are contained in these Provisions and Appendix S. Although the acceptance decision usually focuses on the level of plastic rotation achieved, the tendency for connections to degrade in strength as the deformation level increases is also of concern. This type of behavior can increase both the moment demands from P-Δ effects and the likelihood of frame instability. In the absence of additional information, it is believed that the deterioration in flexural strength from M_{max} at 0.03 radians should be limited to a level that is not below M_p, where M_{max} is the maximum moment recorded in the tests and M_p is the nominal plastic flexural strength based on the specified minimum yield strength F_y as shown in Figure C-9.1. When beam flange buckling or a Reduced Beam Section limits the strength, rather than the connection itself, deterioration to $0.8M_p$ is permitted by exception in Section 9.2b.a.

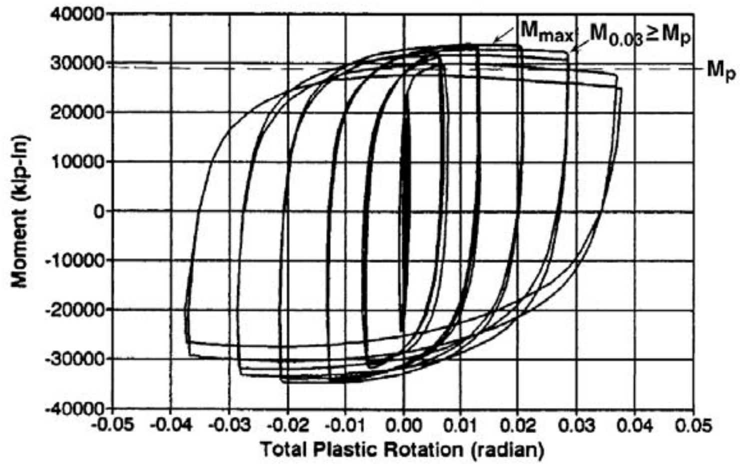


Fig. C-I-9.1. Acceptable strength degradation during hysteretic behavior, per Section 9.2b.

The second exception in Section 9.2b is intended to permit the use of partially restrained (PR) connections. It should also be recognized that truss moment frames can be designed with bottom-chord connections that can deform inelastically and such frames are permitted as SMF if all of the provisions of Section 9 are met.

C9.2c. The required shear strength V_u of the beam-to-column joint is defined as the summation of the factored gravity loads and the shear that results from the required flexural strengths on the two ends of the beam, which can be determined as $1.1R_yF_yZ$. However, in some cases, such as when large gravity loads occur or when panel-zones are weak, rational analysis may indicate that lower combinations of end moments are justified.

C9.3. Panel-zone of Beam-to-Column Connection (Beam web parallel to column web)

Cyclic testing has demonstrated that significant ductility can be obtained through shear yielding in column panel-zones through many cycles of inelastic distortion (Popov et al., 1996; Slutter, 1981; Becker, 1971; Fielding and Huang, 1971; Krawinkler, 1978). Consequently, it is not generally necessary to provide a panel-zone that is capable of developing the full flexural strength of the connected beams if the design strength of the panel-zone can be predicted. However, the usual assumption that Von Mises criterion applies and the shear strength is $0.55F_ydt$ does not match the actual behavior observed in many tests into the inelastic range. Due to the presence of the column flanges, strain hardening and other phenomena, panel-zone shear strengths in excess of F_ydt have been observed. Accordingly, Equation 9-1 accounts for the significant strength contribution of thick column flanges.

Equation 9-1 represents a design strength in the inelastic range and, therefore, is for comparison to factored loads In the 1991 Uniform Building Code (ICBO, 1991), the minimum required panel-zone shear strength was determined by multiplying the service-load panel-zone shear force by 1.85. In these Provisions and in the LRFD Specification, Load Combinations A4-5 and A4-6 are used to determine the required panel-zone shear strength. Because all of the effects of panel-zone yielding may not be positive, ϕ is conservatively specified in these Provisions as 0.75, which results in a reliability that is approximately equivalent to that obtained with the aforementioned provisions in the 1991 Uniform Building Code; ϕ is specified for non-seismic applications as 0.9 in the LRFD Specification.

As an upper limit, the design panel-zone shear strength need not exceed that due to 80 percent of the summation of the expected plastic moments R_yM_p of the beam(s) framing into the panel-zone. The factor of 80 percent is intended to recognize that because of gravity loads and the variation in inflection point locations observed in inelastic analysis, it is unlikely that the full M_p will occur on both sides of a given column at the same time. Additionally, since panel-zone yielding within limits is a relatively benign event, and since web doubler plates are expensive and contribute to possibly undesirable shrinkage, distortion and residual stress conditions, it would be too conservative to use the full summation of M_p.

To minimize shear buckling of the panel-zone during inelastic deformations, the minimum panel-zone thickness is set at one-ninetieth of the sum of its depth and width. Thus, when the column web and web doubler plate(s) each meet the requirements of Equation 9-2, their interconnection with plug welds is not required. Otherwise, the column web and web doubler plate(s) can be interconnected with plug welds as illustrated in Figure C-9.2 and the total panel-zone thickness can be used in Equation 9-2.

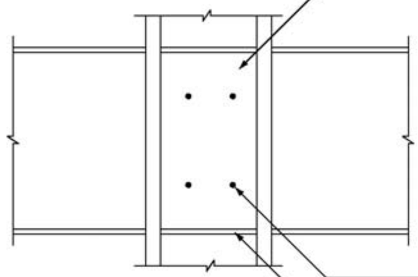


Fig. C-I-9.2. Connecting web doubler plates with plug welds.

In the 1992 AISC Seismic Provisions, it was required that web doubler plates be placed directly against the column web in all cases. In this revision, it is permitted as an alternative to place web doubler plates symmetrically in pairs spaced away from the column web. In the latter configuration, both the web doubler plates and the column web are required to all independently meet Equation 9-2 in order to be considered as effective.

Web doubler plates may extend between top and bottom continuity plates that are welded directly to the column web and web doubler plate or they may extend above and below top and bottom continuity plates that are welded to the doubler plate only. In the former case, the welded joint connecting the continuity plate to the column web and web doubler plate is required to be configured to transmit the proportionate force from the continuity plate to each element of the panel-zone. In the latter case, the welded joint connecting the continuity plate to the web doubler plate is required to be sized to transmit the force from the continuity plate to the web doubler plate and the web doubler plate thickness is required to be selected to transmit this same force; minimize-size fillet welds per LRFD Specification Table J2.4 are used to connect along the column-web edges.

The shear forces transmitted to the web doubler plate from the continuity plates are equilibrated by shear forces along the column-flange edges of the web doubler plate. Because it is anticipated that the panel-zone will yield in a seismic event, the welds connecting the web doubler plate to the column flanges are required to be sized to develop the shear strength of the full web doubler plate thickness. Either a complete-joint-penetration groove-welded joint or a fillet-welded joint can be used as illustrated in Figure C-9.3.

The beneficial role of panel-zone deformation in dissipating energy from earthquakes has been reported in numerous tests as described above. However, recent tests appear to demonstrate that excessive panel-zone deformations may lead to beam flange-to-column flange joint failure at lower than anticipated levels of plastic rotation (Popov et al., 1996) due to local bending of the column flange adjacent to the weld connecting the column to the beam flange. The line between acceptable and excessive panel-zone deformation has not been clearly defined. Therefore at this time no change in the determination of the nominal panel-zone shear strength has been made.

The use of diagonal stiffeners for strengthening and stiffening of the panel-zone has not been adequately tested for low-cycle reversed loading into the inelastic range. Thus no specific recommendations are made at this time for special seismic requirements for this detail.

C9.4. Beam and Column Limitations

To provide for reliable inelastic deformations, the width-thickness ratios of projecting elements should be within those that provide a cross-section that is resistant to local buckling into the inelastic range. Although the width-thickness ratios for compact elements in LRFD Specification Table B5.1 are sufficient to prevent local buckling before the onset of yielding, the available test data suggests that these limits are not adequate for the required inelastic performance in SMF. The limits given in Table I-9-1 are deemed adequate for ductilities to 6 or 7 (Sawyer, 1961; Lay, 1965; Kemp, 1986; Bansal, 1971).

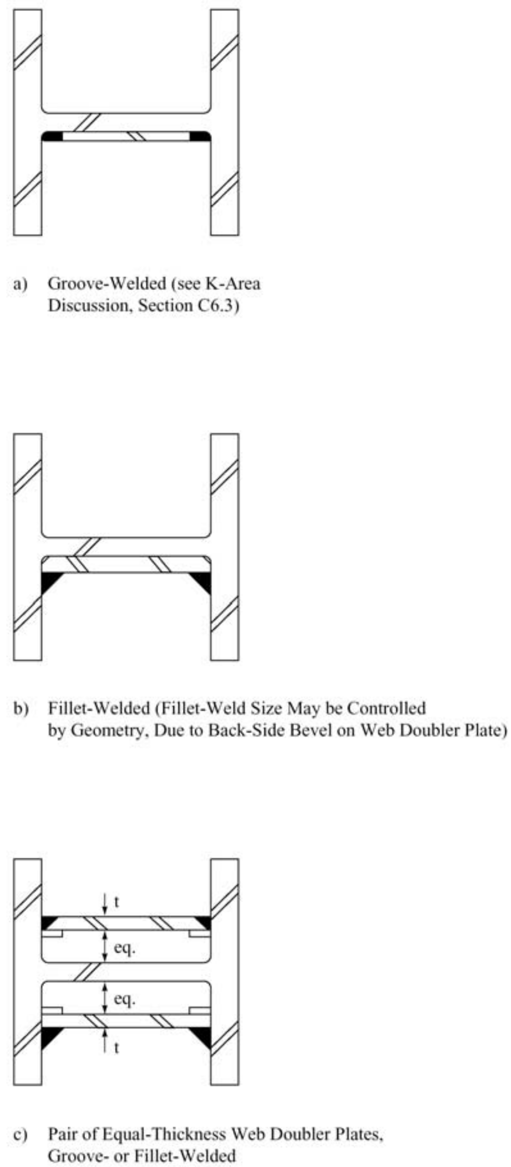


Fig. C-9.3. Web doubler plates.

C9.5. Continuity Plates

When subjected to seismic forces, an interior column (i.e., one with adjacent moment connections to both flanges) in a moment frame receives a tensile flange force on one flange and a compressive flange force on the opposite side. When stiffeners are required, it is normal to place a full-depth transverse stiffener on each side of the column web. As this stiffener provides a load path for the flanges on both sides of the column, it is commonly called a continuity plate. The stiffener also serves as a boundary to the very highly stressed panel-zone. When the formation of a plastic hinge is anticipated adjacent to the column, the required strength is the flange force that is exerted when the full beam plastic moment has been reached, including the effects of overstrength and strain hardening, as well as shear amplification from the hinge location to the column face.

Post-Northridge studies have shown that even when continuity plates of substantial thickness are used, inelastic strains across the weld of the beam flange to the column flange are substantially higher opposite the column web than they are at the flange tips. Some studies have indicated concentrations higher than 4, which can cause the weld stress at the center of the flange to exceed its tensile strength before the flange force exceeds its yield strength based on a uniform average stress. This condition will be exacerbated if relatively thin continuity plates are used or if continuity plates are omitted entirely. For this reason, the use of continuity plates is recommended in all cases unless tests have shown that other design features of a given connection are so effective in reducing or redistributing flange stresses that the connection will work without them.

Given the stress distribution cited above, there is little justification for some of the old rules for sizing and connecting continuity plates, such as selecting its thickness equal to one-half the thickness of the beam flange. On the other hand, the use of excessively thick continuity plates will likely result in large residual stresses, which may similarly be detrimental. Because of the above apparently conflicting concepts, it is judged that continuity plate usage and sizing should be based on tests.

C9.6. Column-Beam Moment Ratio

The strong-column weak-beam (SC/WB) concept is perhaps one of the least-understood seismic provisions in steel design. Some engineers believe that it is formulated to prevent any column flange yielding in a frame and that if such yielding occurs, the column will fail. This is not the case, as tests have shown that yielding of columns in moment frame subassemblages does not reduce the lateral strength at the expected seismic displacement levels.

The SC/WB concept represents more of a global frame concern than a concern at the interconnections of individual beams and columns. Schneider et al. (1991) and Roeder (1987) showed that the real benefit of the SC/WB concept is that the columns are generally strong enough to force flexural yielding in beams in multiple levels of the frame, thereby achieving a higher level of energy dissipation. Weak column frames, on the other hand, are likely to exhibit undesirable response, particularly inelastic weak or soft stories, at those levels with the highest column demand to capacity ratios.

It should be noted that compliance with the SC/WB concept and Equation 9-3 gives no assurance that individual columns will not yield, even when all connection locations in the frame comply. It can be shown with nonlinear analysis that, as the frame deforms inelastically, points of inflection shift and the distribution of moments varies from the idealized condition. Nonetheless, it is believed that yielding of the beams rather than columns will predominate and the desired inelastic performance will be achieved in frames composed of members that meet the requirement in Equation 9-3.

Equation 9-3 is somewhat more complex than the formulation that was used in the 1992 AISC Seismic Provisions wherein the beam/column intersection was idealized as a point at the intersection of the member centerlines. Because post-Northridge beam-to-column moment connections are generally configured to shift the plastic hinge location into the beam away from the column face, a more general formulation was needed. Recognition of potential beam overstrength (see Commentary Section C6.2) is also incorporated into Equation 9-3.

The exceptions wherein framing members need not meet the requirement in Equation 9-3 are given in Sections 9.6a and 9.6b. The compactness requirements in Section 9.4 are required to be met for columns in these exceptions because it is expected that flexural yielding will occur in the columns.

In Section 9.6a, columns with low axial loads that are used in one-story buildings or in the top story of a multi-story building need not meet Equation 9-3 because concerns for inelastic soft or weak stories are of no significance in such cases. Also excepted are prescribed percentages of columns that are low enough that, in the opinion of the Committee, performance will not be undesirable, yet high enough to provide reasonable flexibility to account for conditions where the requirement in Equation 9-3 would be impractical, such as at a large transfer girder.

In Section 9.6b, an exception is provided for columns in levels that are significantly stronger than in the level above since column yielding would therefore be unlikely at that level.

C9.7. Beam-to-Column Connection Restraint

Columns are required to be braced to prevent rotation out of the plane of the moment frame, particularly if inelastic behavior is expected in or adjacent to the beam-to-column connection during high seismic activity.

C9.7a. Restrained Connections: Beam-to-column connections are usually restrained laterally by the floor or roof framing. When this is the case and it can be shown that the column remains elastic outside of the panel-zone, lateral support of the column flanges is required only at the level of the top flanges of the beams. Although arbitrary, the two criteria given to demonstrate that the column remains elastic are reasonable. If it cannot be shown that the column remains elastic, lateral support is required at both the top and bottom beam flanges because of the potential for flexural yielding of the column.

The required strength for lateral support at the beam-to-column connection is 2 percent of the nominal strength of the beam flange. In addition, the element(s) providing lateral support are required to have adequate stiffness to inhibit lateral movement of the column flanges (Bansal, 1971). In some cases, a bracing member will be required for such lateral support. Alternatively, it may be shown that adequate lateral support can be provided by the column web and continuity plates or by the beam flanges.

C9.7b. Unrestrained Connections: Unrestrained connections occur in special cases, such as in two-story frames, at mechanical floors or in atriums and similar architectural spaces. When such connections occur, the potential for out-of-plane buckling at the connection should be minimized. Three provisions are given for the columns to assure that this buckling does not occur.

C9.8. Lateral Support of Beams

The general requirements for lateral support of beams are given in LRFD Specification Chapter F. In moment frames, the beams are nearly always bent in reverse curvature between columns unless one end is pinned. Using a plastic design model as a guide and assuming that the moment at one end of a beam is M_p and a pinned end exists at the other, LRFD Specification Equation F1-1 indicates a maximum distance between points of lateral support of $3{,}600r_y/F_y$. However, there remains the uncertainty of the locations of plastic hinges due to earthquake motions. Consequently, the maximum distance between points of lateral support is conservatively specified as $2{,}500r_y/F_y$ for both top and bottom flanges.

C10. INTERMEDIATE MOMENT FRAMES (IMF)

C10.1. Scope

An Intermediate Moment Frame (IMF) is a new category of moment frame that is intended to provide inelastic rotation capability that is intermediate between that provided by SMF and OMF. It is intended that IMF will not require the larger plastic rotations expected of SMF, because of the use of more or larger framing members than for a comparably designed SMF, or because of use in lower seismic zones. Except for the difference in required connection rotation capacity, the provisions for IMF's are identical to those for SMF's with a few exceptions. Refer to Commentary Section 9 for additional information.

C10.2. Beam-to-Column Joints and Connections

The minimum plastic rotation capability required for IMF is 2 percent while that for SMF is 3 percent. This level of plastic rotation has been established for this type of frame based upon engineering judgment applied to available tests and analytical studies (FEMA, 1995; SAC, 1995d).

C10.8. Lateral Support at Beams

In recognition of the lower anticipated inelastic deformations for IMF, beam flange bracing is permitted to be spaced at wider intervals than those required for SMF. This slightly liberalized requirement will make lateral buckling more likely should larger-than-expected levels of plastic rotation occur.

C11. ORDINARY MOMENT FRAMES (OMF)

C11.1. Scope

Ordinary Moment Frames (OMF) are intended to provide for limited levels of inelastic rotation capability. It is intended that OMF will not require the larger plastic rotations expected of SMF and IMF, because of the use of more or larger framing members than for a comparably designed SMF or IMF, or because of use in lower seismic zones. Because little inelastic action is required, many of the restrictions applied to SMF and IMF are not applied to OMF.

C11.2.Beam-to-Column Joints and Connections

Even though the inelastic rotation demands on OMF are expected to be low, the Northridge Earthquake damage demonstrated that little, if any, inelastic rotational capacity was available in the connection prescribed by the codes prior to 1994. Thus, even for OMF, new connection requirements are needed, and these are provided in this section.

The provisions of this section are intended to provide connections with the capability of at least 0.01 radian cyclic inelastic rotation. In lieu of the specific requirements of this section, the designer may employ connections with tested capability to provide the required rotation.

The specific requirements given for connections are given for both FR and PR moment connections. For FR moment connections, a minimum calculated strength of $1.1R_yM_p$ is required to recognize potential overstrength and strain hardening. Additionally, detailing enhancements are required that have been demonstrated by tests to significantly improve the connection performance over the practices employed before Northridge (Kaufmann et al., 1996; Xue et al., 1996).

These tests consisted of five full-scale dynamic cyclic tests using a W14x311 column (ASTM A572 Grade 50) and W36x150 beam (ASTM A36). In addition, small-scale tension specimens were tested to simulate the welded beam flange to column flange joint in the full-scale tests. These tests were conducted using weld metals with different notch toughness characteristics, different backing bar treatment, different web connections and with or without continuity plates. It was demonstrated that improved performance into the inelastic range can be obtained with the following improvements over the prescriptive pre-Northridge connection detail: (1) the use of notch-tough weld metal; (2) the removal of backing bars, backgouging of the weld root, and rewelding with a reinforcing fillet weld; (3) the use of a welded web connection; and (4) the use of continuity plates.

Some of the connections tested in this series appeared to perform well enough to have qualified for use in SMF. However, at this time, it is judged that such connections may not deliver such performance with a reliability that is acceptable for applications other than OMF.

For information on bolted moment end-plate connections in seismic applications, refer to Meng and Murray (1997).

For information on PR connections, the reader is referred to the literature, including the work of Leon (Leon, 1990; Leon and Ammerman, 1990; Leon and Forcier, 1992).

Selected schematic illustrations of potential strong-axis moment connections are given in Figure C-11.1. A welded beam-to-column moment connection in a strong-axis configuration that is similar to the one tested at Lehigh University is illustrated in Figure C-11.1(d). This detail may be suitable for use in OMF with similar member sizes and other conditions.

C11.3. Continuity Plates

For all welded OMF connections that are not based upon tests, continuity plates are required. See Commentary Section C9.5.

C12. SPECIAL TRUSS MOMENT FRAMES (STMF)

C12.1. Scope

Truss-girder moment frames have often been designed with little or no regard for ductility. Research has shown that such truss moment frames have very poor hysteretic behavior with large, sudden reductions in strength and stiffness due to buckling and fracture of web members prior to or early in the dissipation of energy through inelastic deformations (Itani and Goel, 1991; Goel and Itani, 1994a). The resulting hysteretic degradation as illustrated in Figure C-12.1 results in excessively large story drifts in building frames subjected to earthquake ground motions with peak accelerations on the order of 0.4*g* to 0.5*g*.

The research work led to the development of special truss girders that limit inelastic deformations to a special segment of the truss (Itani and Goel, 1991; Goel and Itani, 1994b; Basha and Goel, 1994). As illustrated in Figure C-12.2, the chords and web members (arranged in an X pattern) of the special segment are designed to withstand large inelastic deformations, while the rest of the structure remains elastic. Special Truss Moment Frames (STMF) have been validated by extensive testing of

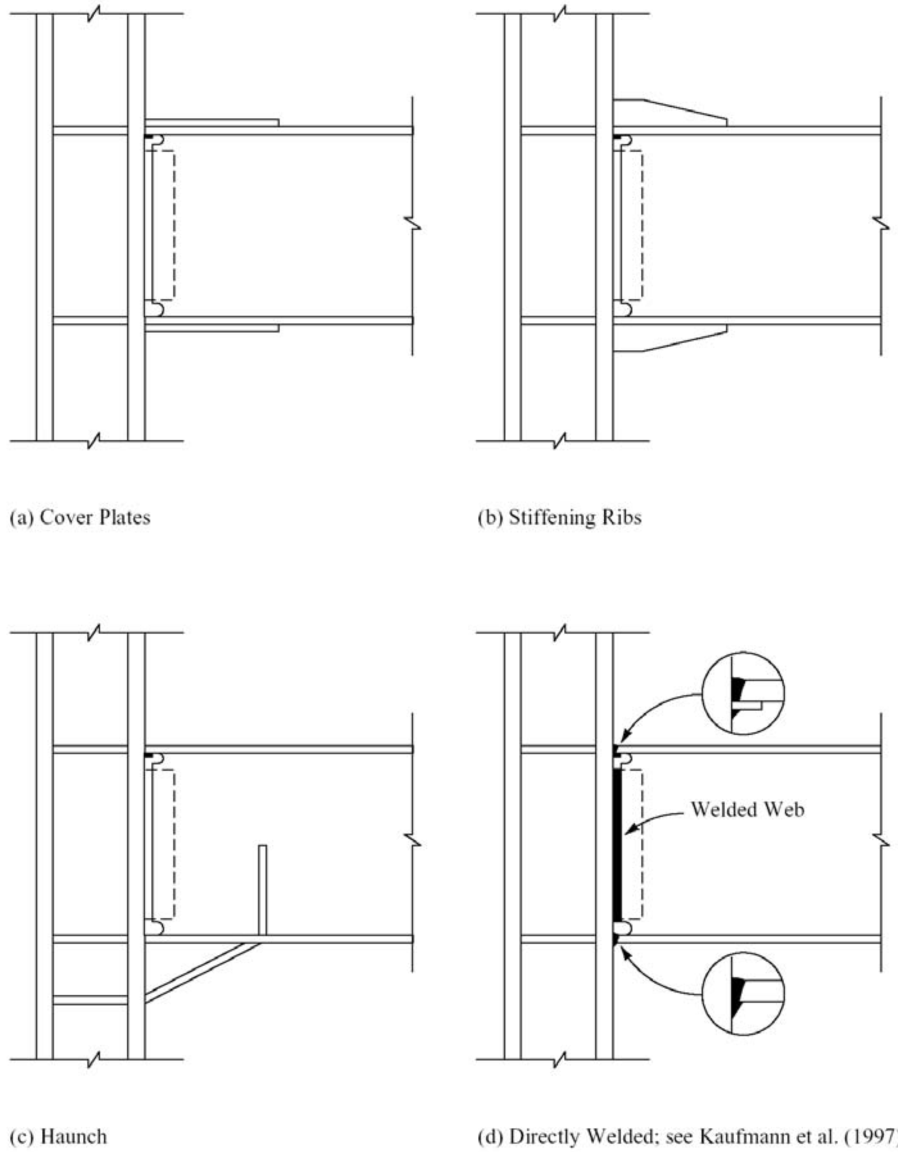


Fig. C-11.1 Schematic illustrations of strong-axis moment connections.

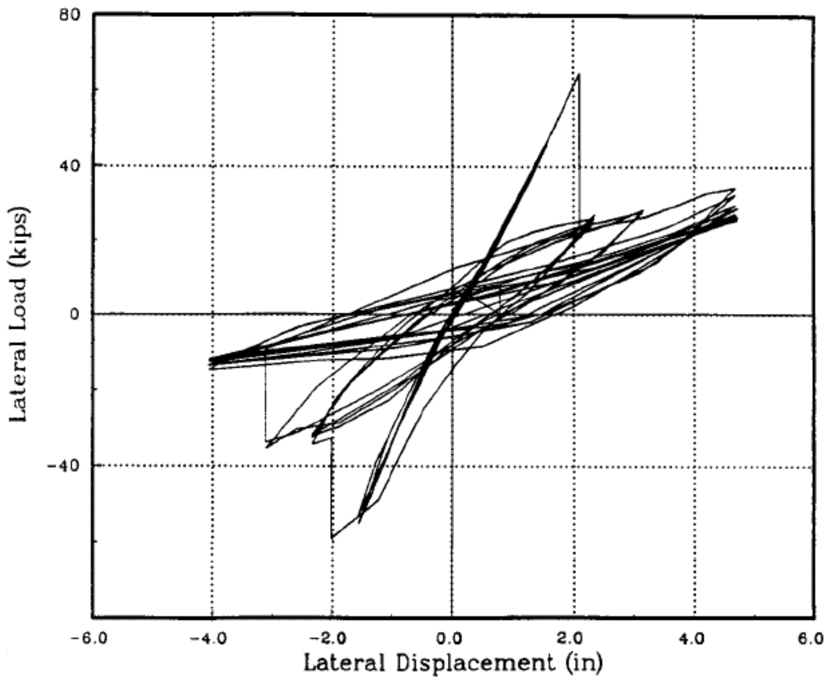


Fig. C-12.1. Strength degradation in undetailed truss girder.

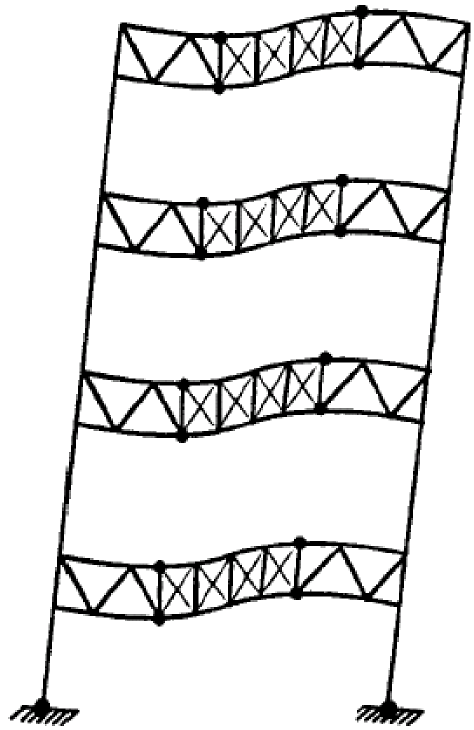


Fig. C-12.2. Cross-braced truss.

full-scale subassemblages with story-high columns and full-span special truss girders. As illustrated in Figure C-12.3, STMF are ductile with stable hysteretic behavior for a large number of cycles up to 3 percent story drifts. Furthermore, inelastic dynamic time history analyses show that STMF response can be significantly superior to that of SMF using solid-web members when both systems are subjected to the same lateral forces.

Because STMF are relatively new and unique, the span length and depth of the truss girders are limited at this time to the range used in the test program.

C12.2. Special Segment

It is desirable to locate the STMF special segment near mid-span of the truss girder because shear due to gravity loads is generally lower in that region. The lower limit on special segment length of 10 percent of the truss span length provides a reasonable limit on the ductility demand, while the upper limit of 50 percent of the truss span length represents more of a practical limit.

Because it is intended that the special segment yield over its full length, no major structural loads should be applied within the length of the special segment. Accordingly, a restrictive upper limit is placed on the axial force in diagonal web members due to gravity loads applied directly within the special segment.

C12.3. Nominal Strength of Special Segment Members

STMF are intended to dissipate energy through flexural yielding of the chord members and axial yielding and buckling of the diagonal web members in the special segment. It is desirable to provide certain minimum shear strength in the special segment through flexural yielding of the chords members and limiting the axial force to a certain maximum value. Plastic analysis can be used to determine the required shear strength of the truss special segments under the factored earthquake load combination.

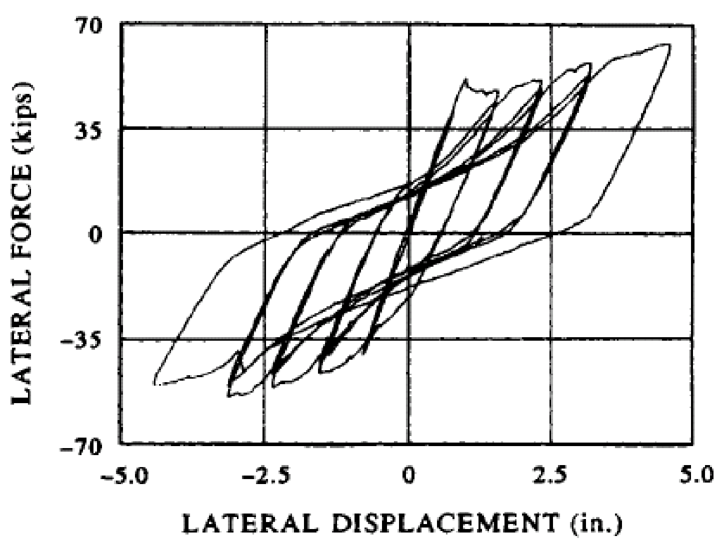


Fig. C-12.3. Hysteretic behavior of STMF.

C12.4. Nominal Strength of Non-Special Segment Members

STMF are required to be designed to maintain elastic behavior of the truss members, columns, and all connections, except for the members of the special segment that are involved in the formation of the yield mechanism. Therefore, all members and connections that are to remain elastic are required to be designed for the combination of gravity loads and lateral forces that are necessary to develop the maximum expected nominal shear strength of the special segment V_{ne} in its fully yielded and strain-hardened state. Thus, Equation 12-1, as formulated, accounts for uncertainties in the actual yield strength of steel and the effects of strain hardening of yielded web members and hinged chord members. It is based upon approximate analysis and test results of special truss girder assemblies that were subjected to story drifts up to 3 percent (Basha and Goel, 1994). Tests on axially loaded members have shown that $0.3P_{nc}$ is representative of the average nominal post-buckling strength under cyclic loading.

C12.5. Compactness

The ductility demand on diagonal web members in the special segment can be rather large. Flat bars are suggested at this time because of their high ductility. Tests (Itani and Goel, 1991) have shown that single angles with width-thickness ratios that are less than $30/\sqrt{F_y}$ also possess adequate ductility for use as web members in an X configuration. Chord members in the special segment are required to be compact cross-sections to facilitate the formation of plastic hinges.

C12.6. Lateral Bracing

The top and bottom chords are required to be laterally braced to provide for the stability of the special segment during cyclic yielding. The lateral bracing limit for flexural members λ_p as specified in the LRFD Specification has been found to be adequate for this purpose.

C13. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

C13.1. Scope

Concentrically braced frames are those braced frames in which the centerlines of members that meet at a joint intersect at a point to form a vertical truss system that resists lateral forces. A few common types of concentrically braced frames are shown in Figure C-13.1, including diagonally braced,

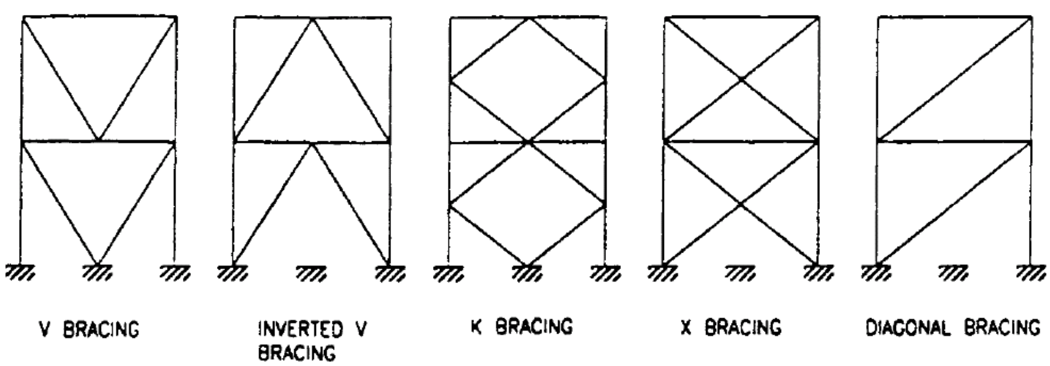


Fig. C-13.1. Examples of concentric bracing configurations.

cross-braced (X), V- braced (or inverted-V-braced) and K-braced configurations. Because of their geometry, concentrically braced frames provide complete truss action with members subjected primarily to axial forces in the elastic range. However, during a moderate to severe earthquake, the bracing members and their connections are expected to undergo significant inelastic deformations into the post-buckling range.

Since the initial adoption of concentrically braced frames into seismic design codes, more emphasis has been placed on increasing brace strength and stiffness, primarily through the use of higher design forces in order to minimize inelastic demand. More recently, requirements for ductility and energy dissipation capability have also been added. Accordingly, provisions for Special Concentrically Braced Frames (SCBF), a new category, have been added. SCBF are intended to exhibit stable and ductile behavior in the event of a major earthquake. Earlier design provisions have been retained for Ordinary Concentrically Braced Frames (OCBF) in Section 14.

During a severe earthquake, bracing members in a concentrically braced frame are subjected to large deformations in cyclic tension and compression into the post-buckling range. As a result, reversed cyclic rotations occur at plastic hinges in much the same way as they do in beams and columns in moment frames. In fact, braces in a typical concentrically braced frame can be expected to yield and buckle at rather moderate story drifts of about 0.3 percent to 0.5 percent. In a severe earthquake, the braces could undergo post-buckling axial deformations 10 to 20 times their yield deformation. In order to survive such large cyclic deformations without premature failure the bracing members and their connections are required to be properly detailed.

Damage during past earthquakes and that observed in laboratory tests of concentrically braced frames has generally resulted from the limited ductility and corresponding brittle failures, which are usually manifested in the fracture of connection elements or bracing members. The lack of compactness in braces results in severe local buckling, the resulting high concentration of flexural strains at these locations and reduced ductility. Braces in concentrically braced frames are subject to severe local buckling, with diminished effectiveness in the nonlinear range at low story drifts. Large story drifts that can result from early brace fractures can impose excessive ductility demands on the beams and columns, or their connections.

Research has demonstrated that concentrically braced frames, with proper configuration, member design and detailing can possess ductility far in excess of that previously ascribed to such systems. Extensive analytical and experimental work by Goel and others has shown that improved design parameters, such as limiting width/thickness ratios (to minimize local buckling), closer spacing of stitches and special design and detailing of end connections greatly improve the post-buckling behavior of concentrically braced frames. The design requirements for SCBF are based on those developments.

Previous requirements for concentrically braced frames sought reliable behavior by limiting global buckling. Cyclic testing of diagonal bracing systems verifies that energy can be dissipated after the onset of global buckling if brittle failures due to local buckling, stability problems and connection fractures are prevented. When properly detailed for ductility as prescribed in these Provisions, diagonal braces can sustain large inelastic cyclic deformations without experiencing premature failures.

Analytical studies (Tang and Goel, 1987; Hassan and Goel, 1991) on bracing systems designed in strict accordance with earlier code requirements for concentrically braced frames predicted brace failures without the development of significant energy dissipation. Failures occurred most often at plastic hinges (local buckling due to lack of compactness) or in the connections. Plastic hinges normally occur at the ends of a brace and at the brace midspan. Analytical models of bracing systems that were designed to ensure stable ductile behavior when subjected to the same ground motion records as the previous concentrically braced frame designs exhibited full and stable hysteresis without fracture. Similar results were observed in full-scale tests by Wallace and Krawinkler (1985) and Tang and Goel (1989).

For double-angle and double-channel braces, closer stitch spacing, in addition to more stringent compactness criteria, is required to achieve improved ductility and energy dissipation. This is especially critical for double-angle and double-channel braces that buckle so that large shear forces are imposed on the stitches. Studies also showed that placement of double angles in a toe-to-toe configuration reduces bending strains and local buckling (Aslani and Goel; 1991).

Many of the failures reported in concentrically braced frames due to strong ground motions have been in the connections. Similarly, cyclic testing of specimens designed and detailed in accordance with typical provisions for concentrically braced frames has produced connection failures (Astaneh et al., 1986). Although typical design practice has been to design connections only for axial loads, good post-buckling response demands that eccentricities be accounted for in the connection design, which should be based upon the maximum forces the connection may be required to resist. Good connection performance can be expected if the effects of brace member cyclic post-buckling behavior are considered (Goel, 1992c).

For brace buckling in the plane of the gusset plates, the end connections should be designed for the full axial load and flexural strength of the brace (Astaneh et al., 1986). Note that a realistic value of K should be used to represent the connection fixity.

For brace buckling out of the plane of single plate gussets, weak-axis bending in the gusset is induced by member end rotations. This results in flexible end conditions with plastic hinges at midspan in addition to the hinges that form in the gusset plate. Satisfactory performance can be ensured by allowing the gusset plate to develop restraint-free plastic rotations. This requires that the free length between the end of the brace and the assumed line of restraint for the gusset be sufficiently long to permit plastic rotations, yet short enough to preclude the occurrence of plate buckling prior to member buckling. A length of two times the plate thickness is recommended (Astaneh et al., 1986). Note that this free distance is measured from the end of the brace to a line that is perpendicular to the brace centerline, drawn from the point on the gusset plate nearest to the brace end that is constrained from out-of-plane rotation. See Figure C-13.2. Alternatively, connections with stiffness in two directions, such as crossed gusset plates, can be detailed. Test results indicate that forcing the plastic hinge to occur in the brace rather than the connection plate results in greater energy dissipation capacity (Lee and Goel, 1987).

Since the stringent design and detailing requirements for SCBF are expected to produce more reliable performance when subjected to high energy demands imposed by severe earthquakes, the design force level has been reduced below that required for OCBF.

Bracing connections should not be configured in such a way that beams or columns of the frame are interrupted to allow for a continuous brace element. This provision is necessary to improve the out-of-plane stability of the bracing system at those connections.

C13.2. Bracing Members

C13.2a. The slenderness (Kl/r) limit has been raised to $1000/\sqrt{F_y}$ for SCBF. The more restrictive limit of $720/\sqrt{F_y}$ as specified for OCBF in Section 14.2a is not necessary when the bracing members are detailed for ductile behavior. Tang and Goel (1989) and Goel and Lee (1992) showed that the post-buckling cyclic fracture life of bracing members generally increases with an increase in slenderness ratio. An upper limit is provided to maintain a reasonable level of compressive strength.

C13.2b. The brace strength reduction factor of 0.8 as specified in Section 14.2b for OCBF has little influence on the seismic response of concentrically braced frames when ductile behavior is ensured as for SCBF.

C13.2c. This provision attempts to balance the tensile and compressive resistance across the width and breadth of the building since the buckling and post-buckling strength of the bracing members in compression can be substantially less than that in tension. Good balance helps prevent the accumulation of inelastic drifts in one direction. An exception is provided for cases where the bracing members are sufficiently oversized to provide essentially elastic response.

C13.2d. Width-thickness ratios of compression elements in bracing members have been reduced to be at or below the requirements for compact sections in order to minimize the detrimental effects of local buckling and subsequent fracture during repeated inelastic cycles. Tests have shown this failure mode to be especially prevalent in rectangular HSS with width-thickness ratios larger than the prescribed limits (Hassan and Goel, 1991; Tang and Goel, 1989).

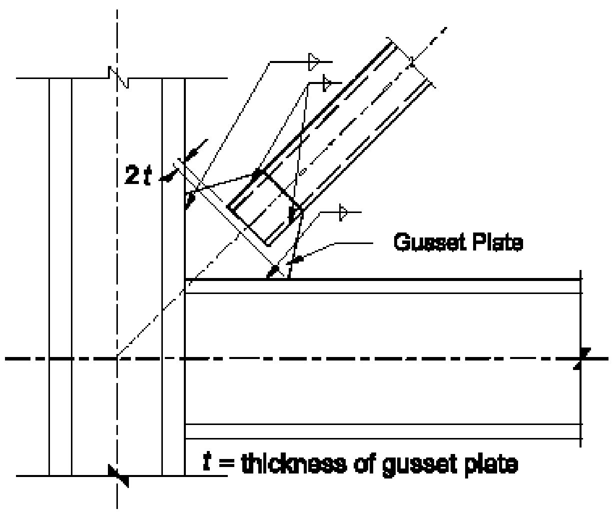


Fig. C-13.2. Brace-to-gusset plate requirement for buckling out-of-plane bracing system.

C13.2e. Closer spacing of stitches and higher stitch strength requirements are specified for built-up bracing members in SCBF (Aslani and Goel, 1991; Xu and Goel, 1990) than those specified in Section 14.2e for OCBF. These are intended to restrict individual element bending between the stitch points and consequent premature fracture of bracing members. Wider spacing is permitted under exception when buckling does not cause shear in the stitches. Bolted stitches are not permitted within one-fourth of the clear brace length as the presence of bolt holes in that region may cause premature fractures due to the formation of plastic hinge in the post-buckling range.

C13.3. Bracing Connections

C13.3a. In concentrically braced frames, the bracing members normally carry most of the seismic story shear, particularly if not used as a part of a dual system. The required strength of bracing connections should be adequate so that failure by out-of-plane gusset buckling or brittle fracture of the connections are not critical failure mechanism.

The minimum of the two criteria, (i.e. the nominal expected axial tension strength of the bracing member and the maximum force that could be generated by the overall system) determines the required strength of both the bracing connection and the beam-to-column connection if it is part of the bracing system. R_y has been added to the first provision to recognize the material overstrength of the member.

C13.3b. Previous requirements considered only net section concerns for bolted connections. These Provisions have been modified to recognize the need to prevent all types of potential local failure in the connections.

C13.3c. Braces that have "fixed" end connections have been shown to dissipate more energy than those that are "pin" connected, because buckling requires the formation of three plastic hinges in the brace. Nonetheless, end connections that can accommodate the rotations associated with brace buckling deformations while maintaining adequate strength have also been shown to have acceptable performance. Testing has demonstrated that where a single gusset plate connection is used, the rotations can be accommodated as long as the brace end is separated by at least two times the gusset thickness from a line perpendicular to the brace axis about which the gusset plate may bend unrestrained by the beam, column, or other brace joints (Astaneh et al., 1986). This condition is illustrated in Figure C-13.2 and provides hysteretic behavior as illustrated in Figure C-13.3. More information on seismic design of gusset plates can be obtained from Astaneh (1998).

Where "fixed"-ended connections are used in one axis with "pinned" connections in the other axis, the effect of the fixity should be considered in determining the critical buckling axis.

C13.4. Special Bracing Configuration Special Requirements

C13.4a. V-braced and Inverted-V-Braced Frames exhibit a special problem that sets them apart from braced frames in which both ends of the braces frame into beam-column joints. Upon continued lateral displacement as the compression brace buckles, its force drops while that in the tension brace continues to increase up to the point of yielding. This creates an unbalanced vertical force on the intersecting beam. In order to prevent undesirable deterioration of lateral strength of the frame, the SCBF provisions require that the beam possess adequate strength to resist this potentially significant post-buckling force redistribution (the unbalanced force) in combination with appropriate gravity loads. Tests have shown that typical bracing members demonstrate a residual post-buckling compressive strength of about 30 percent of the initial compressive strength (Hassan and Goel, 1991). This is the maximum compression force that should be combined with the full yield force of the adjacent tension brace. The full tension force can be expected to be in the range of P_y. The adverse effect of this unbalanced force can be mitigated by using bracing configurations, such as V- and Inverted-V-braces in alternate stories creating an X- configuration over two story modules, or by using a "zipper column" with V- or Inverted-V bracing (Khatib et al., 1988). See Figure C-13.4. Adequate lateral support at the brace-to-beam intersection is necessary in order to prevent adverse effects of possible lateral-torsional buckling of the beam.

The requirements in Sections 13.4a.1 and 13.4a.2 provide for a minimum strength of the beams to support gravity loads in the event of loss of brace capacities.

The limitations of Sections 13.4a.2 and 13.4a.3 need not be applied on beam strength of roof stories, penthouses, and one-story structures as the life safety consequences of excessive beam deformations may not be as severe as for floors.

C13.4b. K-bracing is generally not considered desirable in concentrically braced frames and is prohibited entirely for SCBF because it is considered undesirable to have columns that are subjected to unbalanced lateral forces from the braces, as these forces may contribute to column failures.

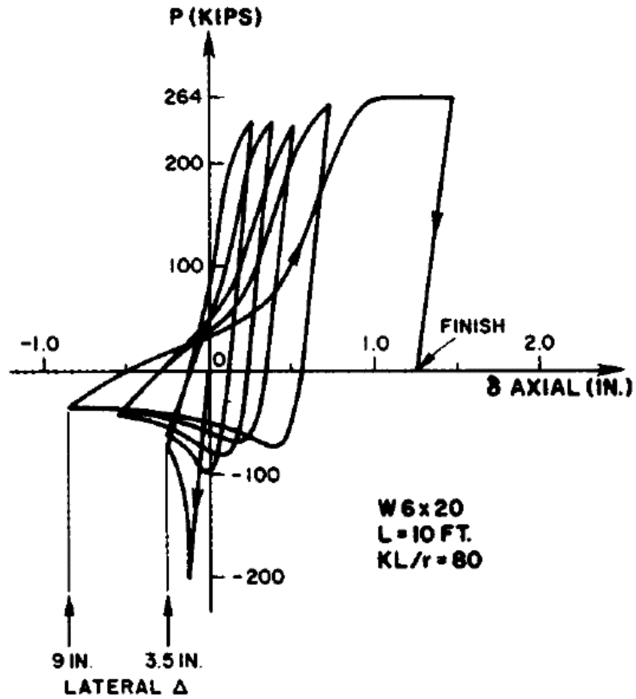


Fig. C-13.3. P-δ diagram for a strut.

C13.5. Columns

In the event of a major earthquake, columns in concentrically braced frames can undergo significant bending beyond the elastic range after buckling and yielding of the braces. Even though their bending strength is not utilized in the design process when elastic design methods are used, columns in SCBF are required to have adequate compactness and shear and flexural strength in order to maintain their lateral strength during large cyclic deformations of the frame. Analytical studies on SCBF that are not part of a dual system have shown that columns can carry as much as 40 percent of the story shear (Tang and Goel, 1987; Hassan and Goel, 1991). When columns are common to both SCBF and SMF in a dual system, their contribution to story shear may be as high as 50 percent. This feature of SCBF greatly helps in making the overall frame hysteretic loops "full" when compared with those of individual bracing members which are generally "pinched" (Hassan and Goel, 1991; Black et al., 1980). See Figure C-13.5.

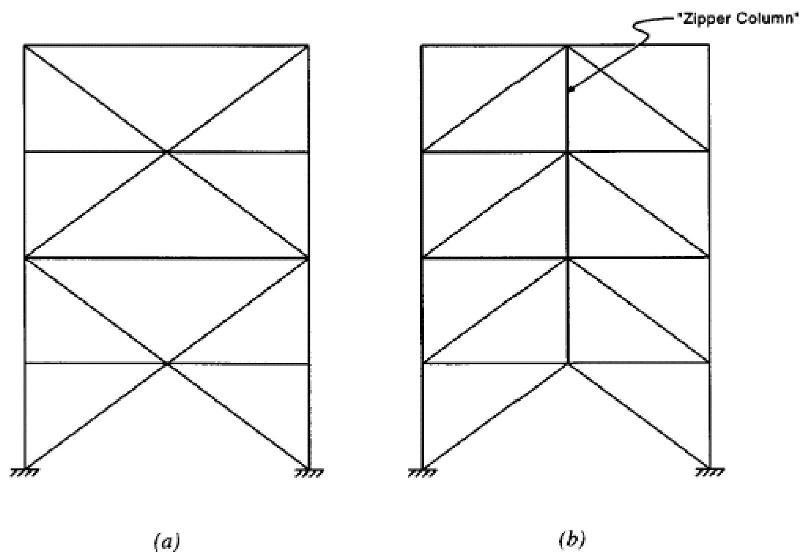


Fig. C-13.4. (a) Two-story X-braced frame, (b) "Zipper-Column" with Inverted-V bracing.

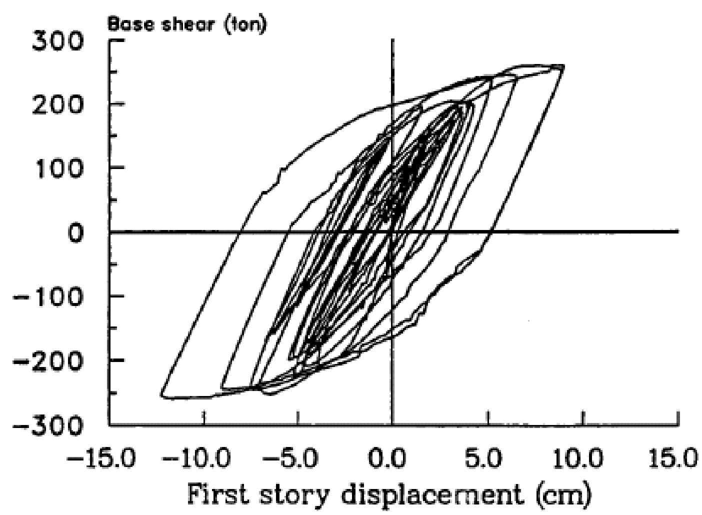


Fig. C-13.5. Base shear vs. story drift of a SCBF.

SCBF column splice requirements for shear are more restrictive than those for SMF.

SCBF requirements should not be waived for low buildings because the R value used is only appropriate with the detailing requirements given here.

C14. ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

C14.1. Scope

These Provisions for Ordinary Concentrically Braced Frames (OCBF) are the same as those that were included in previous editions for concentrically braced frames and contain some but not all of the SCBF detailing requirements that ensure ductile behavior. Generally, the required strengths for OCBF are higher than those for SCBF, which represents an attempt to keep the inelastic deformations from becoming too large in a large seismic event. The comments in this Section are limited to those provisions for OCBF that are different from those for SCBF and the reader is referred to Commentary Section C13 for additional information.

C14.2. Bracing Members

C14.2a. For structures that are taller than two stories, the slenderness ratio Kl/r of the braces is limited to a smaller value of $720/\sqrt{F_y}$ than that for braces in SCBF. Although braces with smaller slenderness will generally dissipate more energy, studies on HSS bracing members have shown that their fracture life and, therefore, total energy dissipation capability may decrease with slenderness ratio (Tang and Goel, 1989; Lee and Goel, 1987).

C14.2b. Due to the cyclic nature of seismic response, the compressive design strength of bracing members is reduced to 80 percent of the value given in LRFD Specification Chapter E. When evaluating the nominal strength of the bracing system for the purpose of determining the maximum load that the bracing can impose on the other elements or system, such as when using Equation (3-1), the reduction for cyclic behavior should not be used as it would underestimate the nominal strength of the bracing system during the early cycles of seismic response.

C14.2e. Adequate shear transfer is required across stitches so that the shear forces associated with the curvatures in the buckled brace can be transferred across the stitches without slip. Welded stitches are recommended. The provision requiring the stitches to be designed for 50 percent of the nominal strength of the individual element is based upon some early test results (Astaneh et al., 1986).

C14.3. Bracing Connections

C14.3a. In order to avoid failure at the brace end connections, the connections should be designed to develop the tensile strength of the brace, or at least the maximum force that can be delivered to the system. It is also considered that minimum force level associated with the amplified loading given by Load Combinations 4-1 and 4-2 can be accepted. These same minimum strength requirements also apply to beam connections that are part of the bracing system.

C14.4. Bracing Configuration

C14.4a. The increase factor of 1.5 for the seismic design force for bracing members in V-Braced or Inverted-V-Braced Frame configurations is carried over from previous editions. Although the increased design force will generally limit post-buckling deformations of the braces, studies have shown that brace buckling can occur at rather moderate story drifts, subjecting the intersecting beams to rather large unbalanced forces as drifts become large (Hassan and Goel, 1991; Tang and Goel, 1989).

C14.4b. In areas of high seismicity where it is envisioned that strong ground motions would cause inelastic response, the K-Braced OCBF is not a desirable system for seismic resistance. Buckling and tension yielding of K-braces creates an unbalanced horizontal force on the columns which can potentially lead to more serious consequences than similar unbalanced force acting on beams in V-Braced or Inverted-V-Braced OCBF.

In buildings that are classified in Seismic Design Categories A, B, and C, K-Braced OCBF are permitted. It is recommended, however, that K-bracing not be used for seismic resistance unless other configurations are impractical.

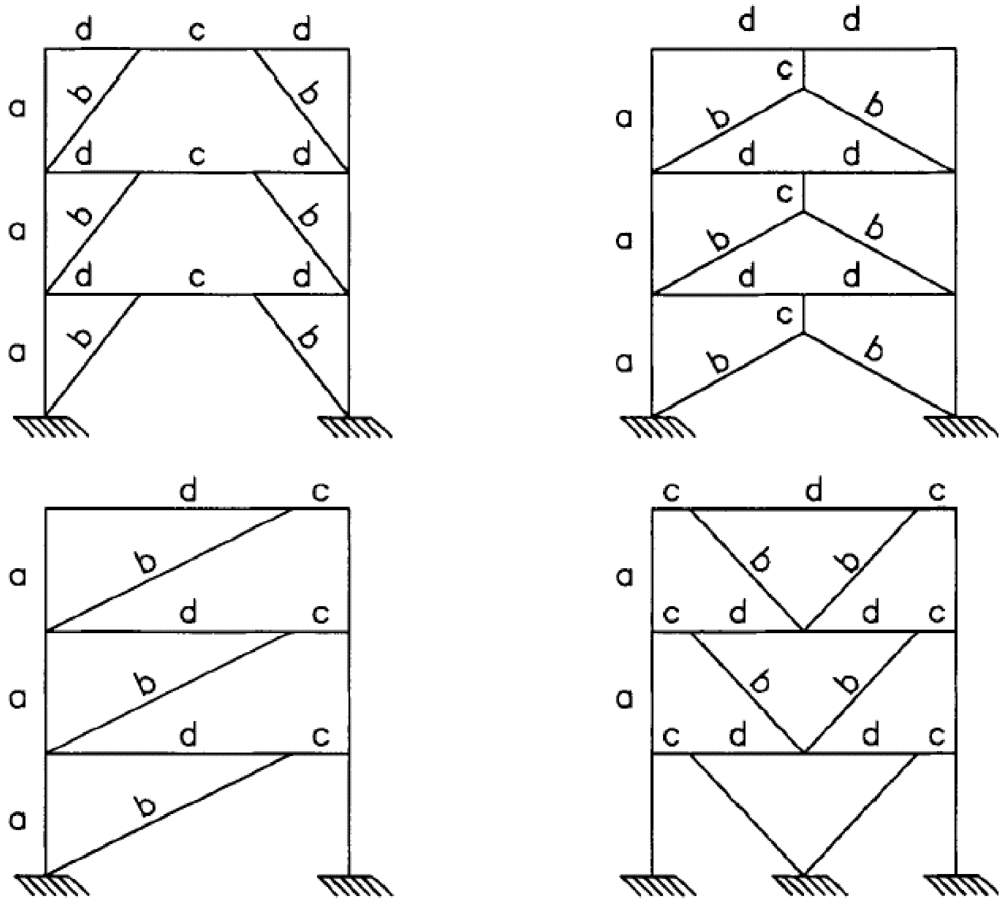
C14.5. Low Buildings

For smaller and less important buildings, the provisions of Sections 14.2 through 14.4 may be waived if the structure has the strength to resist the amplified seismic Load Combinations 4-1 and 4-2. This, for example, would permit tension-only bracing for such structures.

C15. ECCENTRICALLY BRACED FRAMES (EBF)

C15.1. Scope

Research has shown that EBF can provide an elastic stiffness that is comparable to that for SCBF and OCBF, particularly when short Link lengths are used, and excellent ductility and energy dissipation capacity in the inelastic range, comparable to that of SMF (Roeder and Popov; 1978; Libby, 1981; Merovich et al., 1982; Hjelmstad and Popov, 1983; Malley and Popov, 1984; Kasai and Popov, 1986a and 1986b; Ricles and Popov, 1987a and 1987b; Engelhardt and Popov, 1989a and 1989b; Popov et al., 1989). EBF are composed of columns, beams, and braces in which at least one end of each bracing member connects to a beam at a short distance from an adjacent beam-to-brace connection or a beam-to-column connection as illustrated in Figure C-15.1. This short beam segment, called the Link, is intended as the primary zone of inelasticity. These provisions are intended to ensure that cyclic yielding in the Links can occur in a stable manner while the diagonal braces, columns, and portions of the beam outside of the Link remain essentially elastic under the forces that can be generated by fully yielded and strain h-ardened Links.



a = column
b = brace
c = link
d = portion of beam outside of link

Fig. C-15.1. Common types of eccentrically braced frames.

In some bracing arrangements, such as that illustrated in Figure C-15.2 with Links at each end of the brace, Links may not be fully effective. If the upper Link has a significantly lower design shear strength than that for the Link in the story below, the upper Link will deform inelastically and limit the force that can be delivered to the brace and to the lower Link. When this condition occurs the upper Link is termed an active Link and the lower Link is termed an inactive Link. The presence of potentially inactive Links in an EBF increases the difficulty of analysis.

It can be shown with plastic frame analyses that, in some cases, an inactive Link will yield under the combined effect of dead, live and earthquake loads, thereby reducing the frame strength below that expected (Kasai and Popov, 1984). Furthermore, because inactive Links are required to be detailed and constructed as if they were active, and because a predictably inactive Link could otherwise be designed as a pin, the cost of construction is needlessly increased. Thus, an EBF configuration that ensures that all Links will be active, such as that illustrated in Figure C-15.1, is recommended. Further recommendations for the design of EBF are available (Popov et al., 1989).

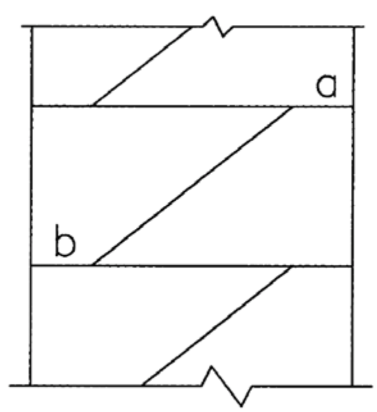
The potential for inelasticity in columns should be avoided in EBF because, when combined with Link inelasticity, a soft story could otherwise result. Accordingly, in Section 7.2, the required axial column strength when $P_u/\phi P_n$ exceeds 0.5 is based upon application of the amplified earthquake load $\Omega_o Q_E$ in Equation 4-1. Furthermore, in Section 15.8, the required strength of columns due to the forces introduced at the connection of a Link and/or brace is based on these forces multiplied by a factor of $1.1R_y$. It should be noted that, in a severe earthquake the formation of plastic hinges at column bases is generally unavoidable.

C15.2. Links

The following general provisions for Links are intended to ensure that stable inelasticity can occur in the Link.

C15.2a. The Link cross-section is required to meet the same width-thickness criteria as is specified for beams in SMF (Table I-9-1).

C15.2b. To ensure the use of steel with proven ductile behavior, the specified minimum yield stress should not exceed 50 ksi.



ϕV_n – link a (active link) < ϕV_n – link b (inactive link)

Fig. C-15.2. EBF – active and inactive link.

C15.2c. The reinforcement of Links with web doubler plates is not permitted as such reinforcement does not fully participate as intended in inelastic deformations. Additionally, beam web penetrations within the Link are not permitted because they adversely affect the desirable yielding of the Link web.

C15.2d. The Link design shear strength ϕV_n is the lesser of that determined from the yield shear or twice the plastic moment divided by the Link length, as dictated by statics assuming equalization of end moments. This design shear strength should then be greater than or equal to the required shear strength determined from the LRFD Specification Load Combinations A4-5 or A4-6.

C15.2e. The effects of axial force on the Link can be ignored if the required axial strength on the Link does not exceed 15 percent of the nominal yield strength of the Link P_y. In general, such an axial load is negligible because the horizontal component of the brace load is transmitted to the beam segment outside of the Link. However, when the framing arrangement is such that larger axial forces can develop in the Link, such as from drag struts or a modified EBF configuration, the additional requirements in Section C15.2f apply and the design shear strength and Link lengths are required to be reduced to ensure stable yielding.

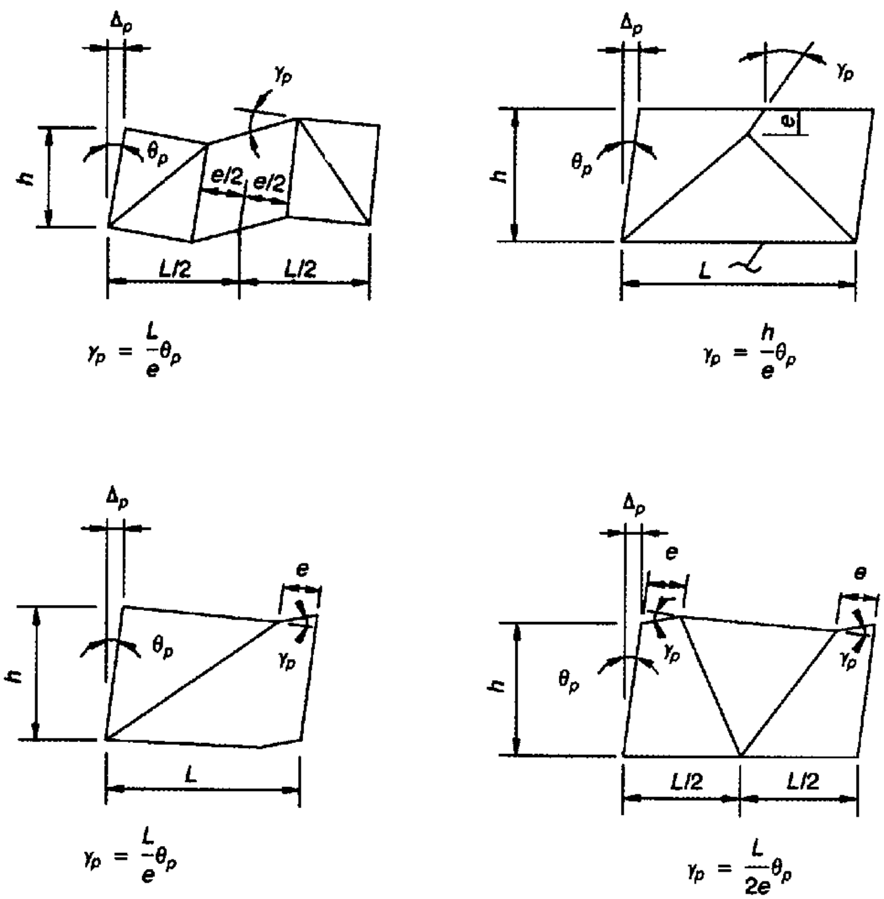
C15.2f. See Commentary Section 15.2e.

C15.2g. The Glossary definition of the Link Rotation Angle in these Provisions has been changed from that used in the 1992 Seismic Provisions, in which the amplified earthquake force was taken as 0.4R times E in calculating the drift. In the 1997 NEHRP Provisions, C_d is used in lieu of 0.4R and results in a higher amplified earthquake force and corresponding drift. The resulting Design Story Drift is a reasonable, though not necessarily maximum, estimation of the total building drift under the Design Earthquake. Accordingly, Link Rotation Angle limits of 8 percent for shear Links and 2 percent for flexural Links were selected from test results to provide a modest reserve rotational capability to accommodate frame deformations beyond those corresponding to the C_d value.

The Link plastic rotation angle can be conservatively estimated by assuming that the EBF bay will deform in a rigid-plastic mechanism as illustrated for various EBF configurations in Figure C-15.3. The plastic rotation angle is determined using a story drift $\Delta_p = \Delta_d - \Delta_e$, where the elastic story drift Δ_e can be taken equal to zero. From geometry, the plastic story drift angle θ_p is then Δ_p/h. Alternatively, the Link plastic rotation angle can be determined more accurately by non-linear elastic-plastic analyses.

For the Inverted-Y-Braced EBF shown in Figure C-15.1, the Glossary definition for the Link Rotation Angle is not technically applicable. Nonetheless, as illustrated in Figure C-15.3, the concept is the same. As usual both ends of the Link are required to be laterally supported.

When the Link length is selected not greater than $1.6M_p/V_p$, shear yielding will dominate the inelastic response. If the Link length is selected greater than $2.6M_p/V_p$, flexural yielding will dominate the inelastic response. For Links lengths intermediate between these values, the inelastic response will occur through some combination of shear and flexural yielding and straight line interpolation is used to determine the appropriate limit.



Δ_v = Story drift determined using base shear v, inches.
Δ_t = Total story drift, inches = $\Delta_v \times e'/e$.
Δ_e = Elastic story drift, inches = Δ_v times the earthquake load factor.
Δ_p = Plastic story drift, inches = $\Delta_t - \Delta_e$ (conservatively, $\Delta_e = 0$).
e = Link length, inches.
h = Story height, inches.
L = Column to column distance, inches.
θ_p = Plastic story drift angle, radians = Δ_p/h.
γ_p = Link rotation angle, radians.

Fig. C-15.3. Link rotation angle.

It has been demonstrated experimentally (Whittaker et al., 1987; Foutch, 1989) as well as analytically (Popov et al., 1989) that Links in the first floor usually undergo the largest inelastic deformation. In extreme cases this may result in a tendency to develop a soft story. The plastic Link rotations tend to attenuate at higher floors, and decrease with the increasing frame periods. Therefore for severe seismic applications, a conservative design for the Links in the first two or three floors is recommended. This can be achieved by increasing the minimum design shear strengths of these Links on the order of 10 percent over that specified in Section 15.2d. Alternatively, a greater degree of conservatism can be obtained by placing vertical members connecting the ends of the Links in a few lower floors.

The use of the framing shown in Figure C-15.1 can be advantageous where the beam-column-brace connections can be designed as simple connections. Welds of the Link flanges are avoided in this kind of framing, but caution is required to enure that the required strength can be provided.

The stiffness of an EBF can be modified to optimize the period of the frame by altering the Link length.

C15.3. Link Stiffeners

A properly detailed and restrained Link web can provide stable, ductile, and predictable behavior under severe cyclic loading. The design of the Link requires close attention to the detailing of the Link web thickness and stiffeners.

C15.3a. Full-depth stiffeners are required at the ends of all Links and serve to transfer the Link shear forces to the reacting elements as well as restrain the Link web against buckling.

C15.3b. The maximum spacing of Link Intermediate Web Stiffeners in shear Links is dependent upon the size of the Link Rotation Angle (Kasai and Popov, 1986b) with a closer spacing required as the rotation angle increases. Flexural Links having lengths greater than $2.6M_p/V_p$ but less than $5M_p/V_p$ are required to have an intermediate stiffener at a distance from the Link end equal to 1.5 times the beam flange width to preclude the possibility of flange local buckling. Links of a length that is between the shear and flexural limits are required to meet the stiffener requirements for both shear and flexural Links. When the Link length exceeds $5M_p/V_p$, Link Intermediate Web Stiffeners are not required. Link Intermediate Web Stiffeners are required to extend full depth in order to effectively resist shear buckling of the web and are required on both sides of the web for Links 25 in. in depth or greater. For Links that are less than 25 in. deep, the stiffener need be on one side only.

This Section was modified slightly from that in the 1992 AISC Seismic Provisions to be compatible with Section 15.2g and to correct minor discrepancies in the stiffener spacing formulas.

C15.3c. All Link stiffeners are required to be fillet welded to the Link web and flanges. The welds to the Link web is required to provide a design strength that is equal to the nominal vertical tensile strength of the stiffener in a section perpendicular to both the plane of the web and the plane of the stiffener or the shear yield strength of the stiffener, whichever is less. The connection to the Link flanges are designed for correspondingly similar forces.

C15.4. Link-to-Column Connections

Previous research indicated that the post-yield behavior of long Links is dominated by large, non-uniformly distributed inelastic flexural strains at the end of the Link, which have led to premature fracture at low inelastic strains in a number of tests. Related research also indicated that the post-yield behavior of short Links is acceptable, being dominated by shear yielding, which at least partially reduces the inelastic flexural strains at the end of the Link. Accordingly, the use of long Links in the Link-to-column configuration was discouraged in the 1992 AISC Seismic Provisions, the use of the Link-to-column EBF configuration with the connection to the weak-axis of a wide-flange column was restricted, and additional restrictions were placed on shear-dominated Link-to-column EBF configurations consistent with the successful tests.

Link-to-column connections in EBF are subject to demands similar to those for beam-to-column connections in moment frames. In many cases they may be subject to larger demands because the inelastic response is confined to a shorter portion of the beam (the Link). Damage to moment connections in the 1994 Northridge earthquake has led to substantial code changes that encourage the physical testing of connections to demonstrate their suitability for seismic applications (see Commentary Sections 9 through 11). Accordingly, the requirements for Link-to-column EBF configurations have been revised to allow two basic alternatives. In the first approach, the expected performance of the Link-to-column connection can be confirmed through approved cyclic testing similar to that for moment connections in Section 9.2a, for a rotation that is at least 20 percent greater than that calculated from the Design Story Drift. Alternatively, shear Links can be placed adjacent to columns with the connection reinforced with haunches or other suitable reinforcement to preclude inelastic action in a transition zone between the Link and column. Such reinforcement is required to maintain nominal elasticity immediately adjacent to the column for the fully yielded and strain-hardened Link strength as defined in Section 15.6a. In lieu of the above, the EBF can be configured to avoid the use of Link-to-column connections entirely.

The LRFD Specification does not explicitly address the column panel-zone design requirements at Link-to-column connections, as little research is available on this issue. However, from research on panel-zones for SMF systems, it is believed that limited yielding of panel-zones in EBF systems would not be detrimental. Pending future research on this topic, it is recommended that the required shear strength of the panel-zone be determined from Equation 9-1 with the flexural demand at the column end of the Link as given by the equations in Commentary Section 15.6a.

C15.5. Lateral Support of the Link

Lateral restraint against out-of-plane displacement and twist is required at the ends of the Link to ensure stable inelastic behavior. The required strength for such lateral support is 6 percent of the nominal strength of the beam flange as determined from physical testing. In typical applications, a composite deck alone can not be counted on to provide adequate lateral support of the Link ends and direct bracing through transverse beams or a suitable alternative is recommended. This provision has been revised to include the R_y factor as described in Section 5.2.

C15.6. Diagonal Brace and Beam Outside of Links

C15.6a. Unlike braces in OCBF, the braces in EBF may be subject to significant bending moments. Accordingly, both the beam and diagonal brace should, in general, be designed as beam-columns to meet the requirements in Section 15.6.

For the beam segment(s) outside of the Link, adequate lateral bracing should be provided to maintain its stability under the axial force and bending moment generated by the Link, as required in Section 15.6d. If the stability of the beam is provided by adequate lateral support, tests have shown that limited yielding of the beam segment is not detrimental to EBF performance, and for some EBF configurations may be unavoidable (Engelhardt and Popov, 1989a). However, the combined flexural strength of the beam and the brace, reduced for the presence of axial force, should be adequate to resist the Link end moment.

For EBF geometries with very small angles between the beam and the brace and/or for EBF with long Links, the requirements in Section 15.6 may result in very heavy braces and, in extreme cases, cover plates on the beams or the use of a built-up member. Thus, EBF with relatively steep braces (brace/beam angles approximately greater than 40 degrees) and short Links are preferable because these difficulties can generally be avoided. A general discussion on design issues related to the beams and braces of EBF is provided in Engelhardt and Popov (1989a), with further details provided in Engelhardt and Popov (1989b).

Inelastic deformations in EBF are restricted to occur primarily in the Links. Accordingly, the diagonal brace and the beam segment(s) outside of the Link should be designed to resist the maximum forces that can be generated by the Link, including consideration of steel overstrength, strain hardening, and the effects of composite floor systems. In EBF research literature, an overstrength factor of 1.5 has generally been applied to the nominal strength of a shear Link to determine the required strength for the brace and the beam. This factor was developed from tests on typical beams with usual flange thicknesses. For Link beams with relatively thick flanges, this factor may need to be increased.

Using this overstrength factor, the brace and beam segment were proportioned with their design strength equal to their nominal strength (i.e., using φ equal to unity), which was considered to be appropriate because the 1.5 overstrength factor represents an extreme loading condition for the beam and brace (Engelhardt and Popov, 1989b). As specified in Section 15.6a, the design strength of the diagonal brace is required to exceed the forces corresponding to R_y times the nominal Link shear strength increased 25 percent for strain hardening. That is, with ϕ equal to 0.85 for axial compression in the brace, the effective overstrength factor (assuming $R_y = 1.1$) becomes 1.25(1.1)/0.85, or about 1.6 for steels with a low variability in F_y and (assuming $R_y = 1.5$) about 2.2 for steels with a high variability. With φ equal to 0.9 for flexure in the beam or diagonal brace, the effective overstrength factor becomes 1.25(1.1)/0.9, or about 1.5, which represents a slight relaxation from the test criterion for steels with a low variability in F_y.

Based on a Link overstrength factor of $1.25R_y$, the required strength of the diagonal brace can be taken as the forces generated by the following values of Link shear and Link end moment:

For $e \leq 2M_p/V_p$, Link shear $= 1.25R_yV_n$
Link end moment $= e(1.25R_yV_n)/2$

For $e > 2M_p/V_p$, Link shear $= 2(1.25R_yM_n)/e$
Link end moment $= 1.25R_yM_n$.

The above equations are based on the assumption that the Link end moments will be equal when the Link deforms plastically. For Links lengths less than or equal to $1.3M_p/V_p$ attached to columns, experiments have shown that Link end moments do not fully equalize during inelastic response (Kasai and Popov, 1986a). For this situation, Link shear and Link end moments can be taken as:

Link shear	$= 1.25R_yV_n$
Link end moment at column	$= 0.8 \text{ x } 1.25R_yM_n$
Link end moment at brace	$= e(1.25R_yV_n) - 0.8M_n$.

The Link shear force will generate axial force in the diagonal brace, and for most EBF configurations, will also generate substantial axial force in the beam segment outside of the Link. The ratio of beam or brace axial force to Link shear force is controlled primarily by the geometry of the EBF and is therefore not affected by inelastic activity within the EBF (Engelhardt and Popov, 1989a). Consequently, this ratio can be determined from an elastic frame analysis and can be used to amplify the beam and brace axial forces to a level that corresponds to the Link shear force specified in the above equations. At the brace end of the Link, the Link end moment will be transferred to the brace and to the beam. If the diagonal brace and its connection remain elastic based on Link overstrength design considerations, some minor inelastic rotation can be tolerated in the beam outside of the Link.

C15.6b. The required strength of the beam outside of the Link has been reduced from that in the 1992 AISC Seismic Provisions.

C15.6c. Typically in EBF design, the intersection of the brace and beam centerlines is located at the end of the Link. However, as permitted in Section 15.6c, the brace connection may be designed with an eccentricity so that the brace and beam centerlines intersect inside of the Link. This eccentricity in the connection generates a moment that is opposite in sign to the Link end moment. Consequently, the value given above for the Link end moment can be reduced by the moment generated by this brace connection eccentricity. This may substantially reduce the moment that will be required to be resisted by the beam and brace, and may be advantageous in design. The intersection of the brace and beam centerlines should not be located outside of the Link, as this increases the bending moment generated in the beam and brace. See Figures C-15.5 and C-15.6.

C15.6d. If the brace connection at the Link is designed as a pin, the beam by itself is required to be adequate to resist the entire Link end moment. This condition normally would occur only in EBF with short Links. If the brace is to resist any portion of the Link end moment, then the brace connection at the Link should be designed as fully restrained, as required in Section 15.6d. Test results on several brace connection details subject to axial force and bending moment are reported in Engelhardt and Popov (1989a).

C15.7. Beam-to-Column Connection

If the arrangement of the EBF system is such that a Link is not adjacent to a column and large axial forces are not present in the beam, a simple connection can be adequate if the connection provides some restraint against torsion in the beam. The magnitude of torsion to be considered is calculated from a pair of perpendicular forces equal to 1.5 percent of the nominal axial flange tensile strength applied in opposite directions on each flange and using the expected yield strength of the flange material.

C15.8. Required Column Strength

To control EBF performance such that Link yielding is the predominant inelastic behavior, an estimate of the maximum actions that can be generated in the columns is required. As the shear strength of the adjoining critical Link is potentially greater than the nominal strength due to strain hardening, the column is required to be designed for the increased moments and axial loads introduced into the column at the connection of a Link or brace at least equal to 1.1 times the expected nominal strength of the Link as given in Section 15.6a. This column strength check is made for EBF in addition to those in Section 8, which is applicable to all systems.

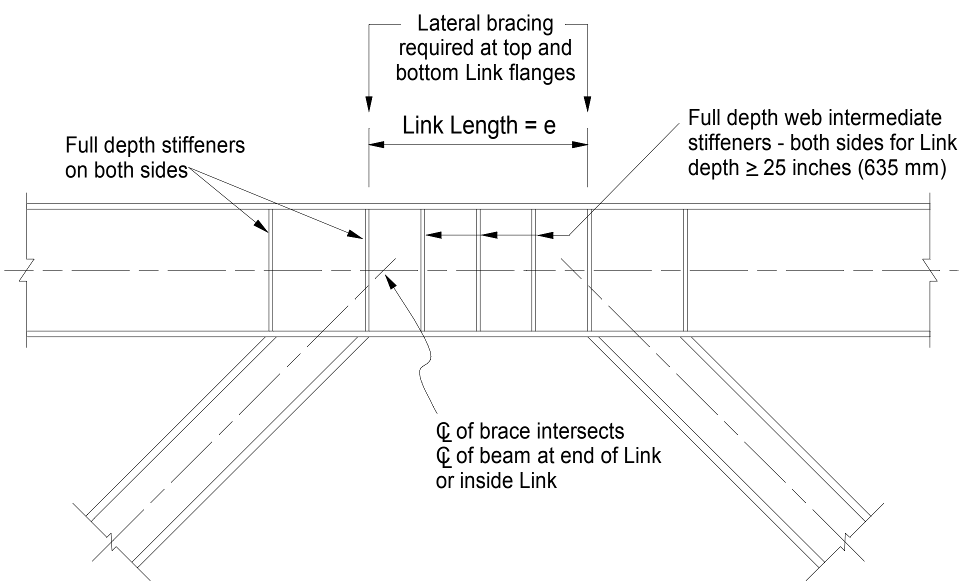


Fig. C-15.5. EBF with W-shape bracing.

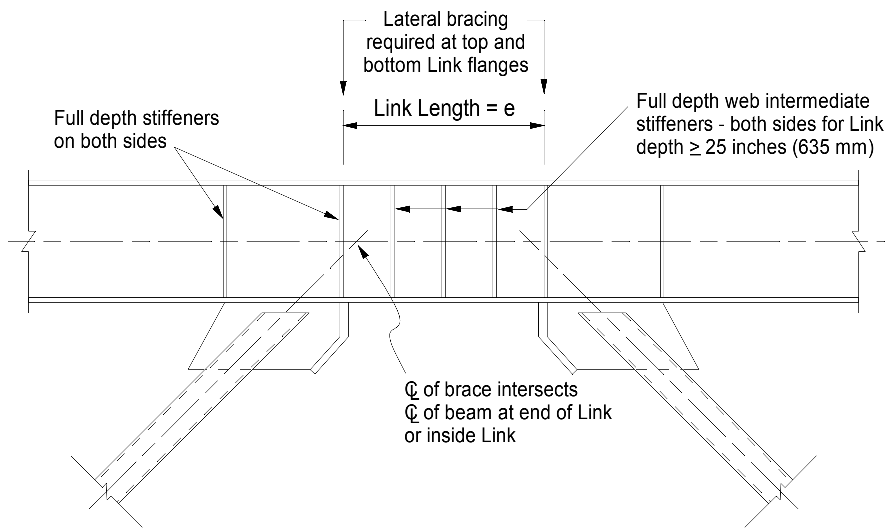


Fig. C-15.6. EBF with HSS bracing.

C16. QUALITY ASSURANCE

To assure ductile seismic response, steel framing is required to meet the quality requirements as appropriate for the various components of the structure. ASCE 7 (ASCE, 1995) provides special requirements for inspection and testing based upon Seismic Design Category. Additionally, these Provisions, the AISC LRFD *Specification for Structural Steel Buildings*, AISC *Code of Standard Practice*, AWS D1.1, and the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* provide acceptance criteria for steel building structures.

These Provisions require that a quality assurance plan be implemented as required by the Engineer of Record. In some cases, the contractor may already have implemented such a plan as part of normal operations, particularly contractors that participate in the AISC Quality Certification Program for steel fabricators. The Engineer of Record should evaluate the quality assurance needs for each project with due consideration of what is already a part of the contractor's quality assurance plan. Where additional needs are identified, such as for innovative connection details or unfamiliar construction methods, supplementary requirements should be specified as appropriate.

Visual inspection prior to, during, and after welding is identified as the primary method used to evaluate the conformance of welded joints to the applicable quality requirements. Joints are examined prior to the commencement of welding to check fit-up, preparation bevels, gaps, alignment, and other variables. During welding, adherence to the WPS is maintained. After the joint is welded, it is then visually inspected to the requirements of AWS D1.1. The subsequent use of other non-destructive examination methods as required by the Engineer of Record is recommended to verify the soundness of welds that are subject to tensile forces as a part of the Seismic Force Resisting Systems described in Sections 9 through 15.

Commentary Section C6.3 indicates that the k-area of rotary-straightened wide-flange columns may have reduced notch toughness. Preliminary recommendations (AISC, 1997) discouraged the placement of welds in this area due to the susceptibility to post-weld cracking that has occurred on past projects. Where such welds are to be placed, it is deemed necessary to perform inspections to verify that such cracking has not occurred. Typically, such inspections would incorporate magnetic particle or dye penetrant testing with acceptance criteria as specified in AWS D1.1. The required frequency of such inspections should be specified in the contract documents.

Appendix S

CS1. SCOPE AND PURPOSE

The development of testing requirements for beam-to-column moment connections was motivated by the widespread occurrence of flange weld fracture in such connections in the 1994 Northridge Earthquake. In order to improve performance of connections in future earthquakes, laboratory testing is required in order to identify potential problems in the design, detailing, materials, or construction methods to be used for the connection. The requirement for testing reflects the view that the behavior of connections under severe cyclic loading cannot be reliably predicted by analytical means only.

It is recognized that testing of connections can be costly and time consuming. Consequently, this Appendix has been written with the most simple testing requirements possible, while still providing reasonable assurance that connections tested in accordance with these Provisions will perform satisfactorily in an actual earthquake. Where conditions in the actual building differ significantly from the test conditions specified in this Appendix, additional testing beyond the requirements herein may be needed to assure satisfactory connection performance. Many of the factors affecting connection performance under earthquake loading are not completely understood. Consequently, testing under conditions that are as close as possible to those found in the actual building will provide for the best representation of expected connection performance.

It is not intended in these Provisions that project-specific connection tests be conducted on a routine basis for building construction projects. In most cases, tests reported in the literature can be used to demonstrate that a connection satisfies the strength and inelastic rotation requirements of these Provisions. Such tests, however, should satisfy the requirements of this Appendix.

Although the provisions in this Appendix predominantly concern the testing of beam-to-column connections in moment frames, they also apply to qualifying cyclic tests of Link-to-column connections in EBF. While there are no reports of failures of Link-to-column connections in the Northridge Earthquake, it cannot be concluded that these similar connections are satisfactory for severe earthquake loading as it appears that few EBF with a Link-to-column configuration were subjected to strong ground motion in this earthquake. Many of the conditions that contributed to poor performance of moment connections in the Northridge Earthquake can also occur in Link-to-column connections in EBF. Consequently, the same testing requirements are applied to both moment connections and to Link-to-column connections.

When developing a test program, the designer should be aware that regulatory agencies may impose additional testing and reporting requirements not covered in this Appendix. Examples of testing guidelines or requirements developed by other organizations or agencies include those published by SAC (FEMA, 1995; FEMA, 1997B), by the ICBO Evaluation Service (ICBO, 1997b), and by the County of Los Angeles (County of Los Angeles Department of Public Works, 1996). Prior to developing a test program, the appropriate regulatory agencies should be consulted to assure the test program meets all applicable requirements. Even when not required, the designer may find the information contained in the foregoing references to be a useful resource in developing a test program.

CS3. DEFINITIONS

InelasticRotation

One of the key parameters measured in a connection test is the inelastic rotation that can be developed in the specimen. For the purpose of demonstrating conformance with requirements in these Provisions, inelastic rotation of a moment connection is required to be computed based on the assumption that all inelastic deformation of a test specimen is concentrated at a single point at the face of the column. In reality, inelastic deformations are distributed over a finite length of the members and/or the connection elements. For many connection types used since the Northridge Earthquake, the portion of the beam subject to yielding is located some distance away from the face of the column. In other cases, yielding may be located within the column panel-zone.

Regardless of where the actual inelastic deformation occurs within the specimen, the inelastic rotation is required to be computed with respect to the face of the column. The purpose of this requirement is to provide a common basis for evaluating connections and to avoid the need for adjusting the acceptance criteria according to different plastic hinge locations. As the actual plastic hinge location is moved away from the face of the column, the inelastic rotation demand at the hinge will increase for the same level of inelastic story drift. However, with the inelastic rotation computed with respect to the face of the column, the inelastic rotation required in these Provisions need not be adjusted for different hinge locations.

The computation of the inelastic rotation requires an analysis of test specimen deformations. Examples of such calculations for moment connections can be found in SAC (1996).

For tests of Link-to-column connections, the key acceptance parameter is the Link inelastic rotation, also referred to in these Provisions as the Link Rotation Angle. The Link Rotation Angle is computed based upon an analysis of test specimen deformations, and can normally be computed as the inelastic portion of the relative end displacement between the ends of the Link, divided by the Link length. Examples of such calculations can be found in Kasai and Popov (1986c), Ricles and Popov (1987) and Engelhardt and Popov (1989a).

CS4. TEST SUBASSEMBLAGE REQUIREMENTS

A variety of different types of subassemblages and test specimens have been used for testing moment connections. A typical subassemblage is planar and consists of a single column with a beam attached on one or both sides of the column. The specimen can be loaded by displacing either the end of the beam(s) or the end of the column. Examples of typical subassemblages for moment connections can be found in the literature, for example in SAC (1996) and Popov et al. (1996).

In these Provisions, test specimens generally need not include a composite slab or the application of axial load to the column. However, such effects may have an influence on connection performance, and their inclusion in a test program should be considered as a means to obtain more realistic test conditions. An example of test subassemblages that include composite floor slabs and/or the application of column axial loads can be found in Popov et al. (1996), Leon et al. (1997), and Tremblay et al. (1997). A variety of other types of subassemblages may be appropriate to simulate specific project conditions, such as a specimen with beams attached in orthogonal directions to a column. A planar bare steel specimen with a single column and a single beam represents the minimum acceptable subassemblage for a moment connection test. However, more extensive and realistic subassemblages that better match actual project conditions should be considered where appropriate and practical, in order to obtain more reliable test results.

CS5. ESSENTIAL TEST VARIABLES

CS5.1. Sources of Inelastic Rotation

This section is intended to assure that the inelastic rotation in the test specimen is developed in the same members and connection elements as anticipated in the prototype. For example, if the prototype connection is designed so that essentially all of the inelastic rotation is developed by yielding of the beam, then the test specimen should be designed and perform in the same way. A test specimen that develops nearly all of its inelastic rotation through yielding of the column panel-zone would not be acceptable to qualify a prototype connection wherein flexural yielding of the beam is expected to be the predominant inelastic action.

Because of normal variations in material properties, the actual location of inelastic action may vary somewhat from that intended in either the test specimen or in the prototype. Consequently, by requiring that only 75 percent of the inelastic rotation occur in the intended elements of the test specimen, some allowance is made for such variations. Thus, for the example above where essentially all of the inelastic rotation in the prototype is expected to be developed by flexural yielding of the beam, at least 75 percent of the total inelastic rotation of the test specimen is required to be developed by flexural yielding of the beam in order to qualify this connection.

For many types of connections, yielding or inelastic deformations may occur in more than a single member or connection element. For example, in some connection types, yielding may occur within the beam, within the column panel-zone, or within both the beam and panel-zone. The actual distribution of yielding between the beam and panel-zone may vary depending upon the beam and column dimensions, web doubler plate thickness, and on the actual yield stress of the beam, column and web doubler plate. Such a connection design can be qualified by running two series of tests: one in which at least 75 percent of the inelastic rotation is developed by beam yielding; and a second in which at least 75 percent of the inelastic rotation is developed by panel-zone yielding. The connection design would then be qualified for any distribution of yielding between the beam and the panel-zone in the prototype.

Satisfying the requirements of this section will require the designer to have a clear understanding of the manner in which a connection develops inelastic rotation.

CS5.2. Size of Members

The intent of this section is that the member sizes used in a test specimen should be, as nearly as practical, a full-scale representation of the member sizes used in the prototype. The purpose of this requirement is to assure that any potentially adverse scale effects are adequately represented in the test specimen. As beams become deeper and heavier, their ability to develop inelastic rotation may be somewhat diminished (Roeder and Foutch, 1996; Blodgett, 1995). Although such scale effects are not yet completely understood, at least two possible detrimental scale effects have been identified. First, as a beam gets deeper, larger inelastic strains are generally required in order to develop the same level of inelastic rotation. Second, the inherent restraint associated with joining thicker materials can affect joint and connection performance. Because of such potentially adverse scale effects, the beam sizes used in test specimens are required to adhere to the limits given in this section.

This section only specifies restrictions on the degree to which test results can be scaled up to deeper or heavier members. There are no restrictions on the degree to which test results can be scaled down to shallower or lighter members. No such restrictions have been imposed in order to avoid excessive testing requirements and because currently available evidence suggests that adverse scale effects are more likely to occur when scaling up test results rather than when scaling down. Nonetheless, caution is advised when using test results on very deep or heavy members to qualify connections for much smaller or lighter members. It is preferable to obtain test results using member sizes that are a realistic representation of the prototype member sizes.

As an example of applying the requirements of this section, consider a test specimen constructed with a W36x150 beam. This specimen could be used to qualify any beam with a depth up to 40 in. (= 36/0.9) and a weight up to 200 lbs/ft (=150/0.75). The limits specified in this section were chosen somewhat arbitrarily based on judgment, as no quantitative research results were available on scale effects.

When choosing a beam size for a test specimen, several other factors should be considered other than just the depth and weight of the section. One of these factors is the width-thickness (b/t) ratios of the beam flange and web. The b/t ratios of the beam may have an important influence on the performance of specimens that develop plastic rotation by flexural yielding of the beam. Beams with high b/t ratios develop local buckling at lower inelastic rotation levels than beams with low b/t ratios. This local buckling causes strength degradation in the beam, and may therefore reduce the force demands on the connection. A beam with very low b/t ratios may experience little if any local buckling, and will therefore subject the connection to higher moments. On the other hand, the beam with high b/t ratios will experience highly localized deformations at local flange and web buckles, which may in turn initiate a fracture. Consequently, it is desirable to test beams over a range of different b/t ratios in order to evaluate these effects.

No specific restrictions are placed on the size of columns used in test specimens in order to avoid excessively burdensome testing requirements. The column size is chosen, however, to produce inelastic deformation in the appropriate elements of the specimen, according to the requirements of Section S5.1. Despite the lack of specific restrictions, it is preferable to choose a column size that provides a realistic representation of the column sizes in the prototype.

CS5.5. Material Strength

The actual yield strength of structural steel can be considerably greater than its specified minimum value. Higher levels of actual yield stress in members that supply inelastic rotation by yielding can be detrimental to connection performance by developing larger forces at the connection prior to yielding. For example, consider a connection design in which inelastic rotation is developed by yielding of the beam, and the beam has been specified to be of ASTM A36 steel. If the beam has an actual yield stress of 55 ksi, the connection is required to resist a moment that is 50 percent higher than if the beam had an actual yield stress of 36 ksi. Consequently, this section requires that the materials used for the test specimen represent this possible overstrength condition, as this will provide for the most severe test of the connection.

As an example of applying these provisions, consider again a test specimen in which inelastic rotation is intended to be developed by yielding of the beam. In order to qualify this connection for ASTM A36 beams, the test beam is required to have a yield stress of at least 46 ksi (= $0.85F_{ye}$ for ASTM A36). This minimum yield strength is required to be exhibited by both the web and flanges of the test beam.

CS5.6. Welds

The intent of these Provisions is to ensure that the welds on the test specimen replicate the welds on the prototype as closely as practicable. Accordingly, it is required that the welding parameters, such as current and voltage, be within the range established by the filler metal manufacturer. Other essential variables, such as steel grade, type of joint, root opening, included angle and preheat level, are required to be in accordance with AWS D1.1.

CS6. LOADING HISTORY

The loading sequence specified in this section is identical to that specified in ATC-24, "Guidelines for Cyclic Seismic Testing of Components of Steel Structures," (ATC, 1992). This document should be consulted for further details of the required loading sequence. Additional displacement increments or additional cycles of loading beyond those specified in Section S6.3 are permitted.

Dynamically applied loads are not required in these Provisions. The use of slowly applied cyclic loads, as typically reported in the literature for connection tests, are acceptable for the purposes of these Provisions. It is recognized that dynamic loading can considerably increase the cost of testing, and that few laboratory facilities have the capability to dynamically load very large scale test specimens. Furthermore, the available research on dynamic loading effects on steel connections has not demonstrated a compelling need for dynamic testing. Nonetheless, applying the required loading sequence dynamically, using loading rates typical of actual earthquake loading, will likely provide a better indication of the expected performance of the connection, and should be considered where possible.

As an alternative to the loading sequence specified in Section S6.3, the SAC loading protocol (SAC, 1997), is considered acceptable. It should be noted that the control variable in the SAC protocol is total drift, rather than plastic rotation as is used in Section S6.3. Modification of the acceptance criteria will be required in order to account for the elastic portion of the specimen displacements. also, for structures located in the near field to causative faults, as defined in ICBO (1997a), loading sequences that focus on the response to near-field ground motions are permitted to be used in lieu of the Basic Loading Sequence. SAC has generated such a loading sequence based upon extensive nonlinear building analyses.

CS8. MATERIALS TESTING REQUIREMENTS

Tension testing is required for the beam, column, and critical connection elements of the test specimen. These tests are required to demonstrate conformance with the requirements of Section S5.5, and to permit proper analysis of test specimen response. Tension test results reported on certified mill test reports are not permitted to be used for this purpose. Yield stress values reported on a certified mill test report may not adequately represent the actual yield strength of the test specimen members. Variations are possible due to material sampling locations and tension test methods used for certified mill test reports.

ASTM standards for tension testing permit the reported yield stress to be taken as the upper yield point. However, for steel members subject to large cyclic inelastic strains, the upper yield point can provide a misleading representation of the actual material behavior. Thus, while an upper yield point is permitted by ASTM, it is not permitted for the purposes of this Section. Determination of yield stress using the 0.2 percent strain offset method is required in this Appendix.

Only tension tests are required in this section. Additional materials testing, however, can sometimes be a valuable aid for interpreting and extrapolating test results. Examples of additional tests which may be useful in certain cases include Charpy V-Notch tests, hardness tests, chemical analysis, and others. Consideration should be given to additional materials testing, where appropriate.

CS10. ACCEPTANCE CRITERIA

A minimum of two tests is required for each condition in the prototype in which the variables listed in Section S5 remain unchanged. The designer is cautioned, however, that two tests, in general, cannot provide a thorough assessment of the capabilities, limitations, and reliability of a connection. Thus, where possible, it is highly desirable to obtain additional test data to permit a better evaluation of the expected response of a connection to earthquake loading. Further, when evaluating the suitability of a proposed connection, it is advisable to consider a broader range of issues other than just inelastic rotation capacity. One factor to consider is the controlling failure mode after the required inelastic rotation has been achieved. For example, a connection that slowly deteriorates in strength due to local buckling may be preferable to a connection that exhibits a more brittle failure mode such as fracture of a weld, fracture of a beam flange, etc., even though both connections achieved the required inelastic rotation. In addition, the designer should also carefully consider the implications of unsuccessful tests. For example, consider a situation where five tests were run on a particular type of connection, two tests successfully met the acceptance criteria, but the other three failed prematurely. This connection could presumably be qualified under these Provisions, since two successful tests are required. Clearly, however, the number of failed tests indicates potential problems with the reliability of the connection. On the other hand, the failure of a tested connection in the laboratory should not, by itself, eliminate that connection from further consideration. As long as the causes of the failure are understood and corrected, and the connection is successfully retested, the connection may be quite acceptable. Thus, while the acceptance criteria in these Provisions have intentionally been kept simple, the choice of a safe, reliable and economical connection still requires considerable judgment.

PART II—Composite Structural Steel and Reinforced Concrete Buildings

C1. SCOPE

These Provisions for the seismic design of composite structural steel and reinforced concrete buildings are based upon the 1994 NEHRP Provisions (FEMA, 1994) and subsequent modifications made in the 1997 NEHRP Provisions (FEMA, 1997a). Chapter 10 of the 1997 NEHRP Provisions references these provisions for detailing and design requirements for composite structures. It is anticipated that the 2000 IBC (ICC, 1997), which is currently in preparation, will similarly reference these Provisions. Since composite systems are assemblies of steel and concrete components, Part I of these Provisions, the LRFD Specification (AISC, 1993) and ACI 318 (ACI, 1995), form an important basis for Part II.

The available research demonstrates that properly detailed composite members and connections can perform reliably when subjected to seismic ground motions. However, there is at present limited experience with composite building systems subjected to extreme seismic forces and many of the recommendations herein are necessarily of a conservative and/or qualitative nature. Careful attention to all aspects of the design is necessary, particularly the general building layout and detailing of members and connections. Composite connection details are illustrated throughout this Commentary to convey the basic character of the composite systems. However, these details should not necessarily be treated as design standards and the reader is strongly encouraged to refer to the cited references for more specific information on the design of composite connections. Additionally, refer to Viest et al. (1997).

The design and construction of composite elements and systems continues to evolve in practice. With further experience and research, it is expected that these provisions can be better quantified, refined and expanded. Nonetheless, these Provisions are not intended to limit the application of new systems, except where explicitly stated, for which testing and analysis demonstrates that the structure has adequate strength, ductility, and toughness.

It is generally anticipated that the overall behavior of the composite systems herein will be similar to that for counterpart structural steel systems or reinforced concrete systems and that inelastic deformations will occur in conventional ways, such as flexural yielding of beams in FR moment frames or axial yielding and/or buckling of braces in braced frames. However, differential stiffness between steel and concrete elements is more significant in the calculation of internal forces and deformations of composite systems than for structural steel only or reinforced concrete only systems. For example, deformations in reinforced concrete elements can vary considerably due to the effects of cracking.

When systems have both ductile and non-ductile elements, the relative stiffness of each should be properly modeled; the ductile elements can deform inelastically while the non-ductile elements remain nominally elastic. When using elastic analysis, member stiffness should be reduced to account for the degree of cracking at the onset of significant yielding in the structure. Additionally, it is necessary to account for material overstrength that may alter relative strength and stiffness.

C2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The specifications, codes and standards that are referenced in Part II are listed with the appropriate revision date that was used in the development of Part II, except those that are already listed in Part I.

C3. SEISMIC DESIGN CATEGORIES

See Part I Commentary Section C3.

C4. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

In general, requirements for loads and load combinations for composite structures are similar to those described in Part I Section C4. However, the 1997 NEHRP Provisions is currently the only code or standard that includes specific seismic loading criteria for these new composite structures. As indicated above, it is anticipated that the 2000 IBC (ICC, 1997) will include seismic loading provisions similar to those in the 1997 NEHRP Provisions.

The calculation of seismic forces for composite systems per the 1997 NEHRP Provisions is the same as is described for steel structures in Part I Commentary Section C4. Table II-C4-1 lists the seismic response modification factors R and C_d for the 1997 NEHRP Provisions. The values in Table II-C4-1 are predicated upon meeting the design and detailing requirements for the composite systems as specified in these provisions. Overstrength factors for the composite systems given in Table II-4-1 of these Provisions are the same as those specified in the 1997 NEHRP Provisions.

ACI 318 Appendix C has been included by reference to facilitate the proportioning of building structures that include members made of steel and concrete. When reinforced concrete members are proportioned using the minimum design loads contained in LRFD Specification Section A4.1, which is consistent with those in ASCE 7 (ASCE, 1995), the strength reduction factors ϕ in ACI 318 Appendix C should be used in lieu of those in ACI 318 Chapter 9.

The seismic response modification factors R and C_d for composite systems specified by the 1997 NEHRP Provisions are similar to those for comparable systems of steel and reinforced concrete. This is based on the fact that, when carefully designed and detailed according to these provisions, the overall inelastic response for composite systems should be similar to comparable steel and reinforced concrete systems. Therefore, in Building Codes where specific loading requirements are not specified for composite systems, appropriate values for the seismic response factors can be inferred from specified values for steel and/or reinforced concrete systems.

TABLE II-C4-1
DESIGN FACTORS FOR COMPOSITE SYSTEMS

BASIC STRUCTURAL SYSTEM AND SEISMIC FORCE RESISTING SYSTEM	*R*	C_d
Systems designed and detailed to meet the requirements of both the LRFD Specification and Part I:		
Braced Frame Systems:		
Composite Concentrically Braced Frame (C-CBF)	5	4½
Ordinary Composite Braced Frames (C-OBF)	3	3
Composite Eccentrically Braced Frames (C-EBF)	8	4
Shear Wall Systems:		
Composite Steel Plate Shear Walls (C-SPW)	6½	5½
Special Reinforced Concrete Shear Walls Composite with Steel Elements (C-SRCW)	6	5
Ordinary Reinforced Concrete Shear Walls Composite with Steel Elements (C-ORCW)	5	4½
Moment Frame Systems:		
Composite Special Moment Frames (C-SMF)	8	5½
Composite Intermediate Moment Frames (C-IMF)	5	4½
Composite Partially Restrained Moment Frame (C-PRMF)	6	5½
Composite Ordinary Moment Frames (C-OMF)	3	2½
Dual Systems with SMF capable of resisting 25 percent of *V*:		
Composite Concentrically Braced Frames (C-CBF)	6	5
Composite Eccentrically Braced Frames (C-EBF)	8	4
Composite Steel Plate Shear Walls (C-SPW)	8	6½
Special Reinforced Concrete Shear Walls Composite with Steel Elements (C-SRCW)	8	6½
Ordinary Reinforced Concrete Shear Walls Composite with Steel Elements (C-ORCW)	7	6
Dual Systems with IMF capable of resisting 25 percent of *V*:		
Composite Concentrically Braced Frame (C-CBF)	5	4½
Composite Ordinary Braced Frame (C-OBF)	4	3
Ordinary Reinforced Concrete Shear Walls Composite with Steel Elements (C-ORCW)	5½	4½

C5. MATERIALS

The limitations in Section 5.1 on structural steel grades used with Part II requirements are the same as those given in Part I. The limitations in Section 5.2 on specified concrete compressive strength in composite members are the same as those given in LRFD Specification Chapter I and ACI 318 Chapter 21. While these limitations are particularly appropriate for construction in Seismic Design Categories D and higher, they apply in any Seismic Design Category when systems are designed with the assumption that inelastic ductility will be present.

C6. COMPOSITE MEMBERS

C6.1. Scope

These Provisions address the seismic design requirements that should be applied in addition to the basic design requirements for gravity and wind loading.

C6.2. Composite Floor and Roof Slabs

In composite construction, floor and roof slabs typically consist of either composite or non-composite metal deck slabs that are connected to the structural framing to provide an in-plane composite diaphragm that collects and distributes seismic forces. Generally, composite action is distinguished from non-composite action on the basis of the out-of-plane shear and flexural behavior and design assumptions.

Composite metal deck slabs are those for which the concrete fill and metal deck work together to resist out-of-plane bending and out-of-plane shear. Flexural strength design procedures and codes of practice for such slabs are well established (ASCE, 1995; ASCE, 1991a and 1991b; AISI, 1996; SDI, 1993).

Non-composite metal deck slabs are one-way or two-way reinforced concrete slabs for which the metal deck acts as formwork during construction, but is not relied upon for composite action. Non-composite metal deck slabs, particularly those used as roofs, can be formed with metal deck and overlaid with insulating concrete fill that is not relied upon for out-of-plane strength and stiffness. Whether or not the slab is designed for composite out-of-plane action, the concrete fill inhibits buckling of the metal deck, increasing the in-plane strength and stiffness of the diaphragm over that of the bare steel deck.

The diaphragm should be designed to collect and distribute seismic forces to the Seismic Force Resisting System. In some cases, forces from other floors should also be included, such as at a level where a change in the structural stiffness results in a redistribution. Recommended diaphragm (in-plane) shear strength and stiffness values for metal deck and composite diaphragms are available for design from industry sources that are based upon tests and recommended by regulatory agencies (Vulcraft, 1990; SDI, 1987; NES, *(biannual review); US Armed Services, 1982; ICBO, *(biannual review); Naeim, 1989). In addition, some recent research on composite diaphragms has been reported (Easterling and Porter, 1994).

As the thickness of concrete over the steel deck is increased, the shear strength can approach that for a concrete slab of the same thickness. For example, in composite floor deck diaphragms having cover depths between 2 in. and 6 in., measured shear stresses on the order of $3.5\sqrt{f_c'}$ (where $\sqrt{f_c'}$ and f'_c are in units of psi) have been reported. In such cases, the diaphragm strength of concrete metal deck slabs can be conservatively based on the principles of reinforced concrete design (ACI, 1995) using the concrete and reinforcement above the metal deck ribs and ignoring the beneficial effect of the concrete in the flutes.

The shear forces are required to be transferred through welds and/or shear devices in the collector and boundary elements. Fasteners between the diaphragm and the steel framing should be capable of transferring forces using either welds or shear devices. Where concrete fill is present, it is generally advisable to use mechanical devices such as headed shear stud connectors to transfer diaphragm forces between the slab and collector/boundary elements, particularly in complex shaped diaphragms with discontinuities. However, in low-rise buildings without abrupt discontinuities in the shape of the diaphragms or in the Seismic Force Resisting System, the standard metal deck attachment procedures may be acceptable.

C6.3. Composite Beams

These provisions apply only to composite beams that are part of the Seismic Force Resisting System.

When the design of a composite beam satisfies Equation 6-1, the strain in the steel at the extreme fiber will be at least five times the tensile yield strain prior to concrete crushing at strain equal to 0.003. It is expected that this ductility limit will control the beam geometry only in extreme beam/slab proportions.

While these Provisions permit the design of composite beams based solely upon the requirements in the LRFD Specification, the effects of reversed cyclic loading on the strength and stiffness of shear studs should be considered. This is particularly important for C-SMF where the design forces are calculated assuming large member ductility and toughness. In the absence of test data to support specific requirements in these Provisions, the following special measures should be considered in C-SMF: (1) implementation of an inspection and quality assurance plan to insure proper welding of shear stud connectors to the beams; and (2) use of additional shear stud connectors beyond those required in the LRFD Specification in regions of the beams where plastic hinging is expected.

C6.4. Reinforced-concrete-encased Composite Columns

The basic requirements and limitations for determining the design strength of encased composite columns are the same as those in the LRFD Specification. Additional requirements for reinforcing bar details of composite columns that are not covered in the LRFD Specification are included based on provisions in ACI 318.

Composite columns can be an ideal solution for use in seismic regions because of their inherent structural redundancy. For example, if a composite column is designed such that the structural steel can carry most or all of the dead load acting alone, then an extra degree of protection and safety is afforded, even in a severe earthquake where excursions into the inelastic range can be expected to deteriorate concrete cover and buckle reinforcing steel. However, as with any column of concrete and reinforcement, the designer should be aware of the constructability concerns with the placement of reinforcement and potential for congestion. This is particularly true at beam-to-column connections where potential interference between a steel spandrel beam, a perpendicular floor beam, vertical bars, joint ties, and shear stud connectors can cause difficulty in reinforcing bar placement and a potential for honeycombing of the concrete.

Seismic detailing requirements for composite columns are specified in the following three categories: ordinary, intermediate, and special. The required level of detailing is specified in these Provisions for seismic systems in Sections 8 through 17. The ordinary detailing requirements of Section 6.4a are intended as basic requirements for all cases. Intermediate requirements are intended for seismic systems permitted in Seismic Design Category C, and special requirements are intended for seismic systems permitted in Seismic Design Categories D and above.

C6.4a. Ordinary Seismic System Requirements.

These requirements are intended to supplement the basic requirements of the LRFD Specification for encased composite columns in all Seismic Design Categories.

1. Specific instructions are given for the determination of the nominal shear strength in concrete encased steel composite members including assignment of some shear to the reinforced concrete encasement. Examples for determining the effective shear width

b_w of the reinforced concrete encasement are illustrated in Figure C-6.1. These provisions exclude any strength V_c assigned to concrete alone (Furlong, 1997).

2. Currently no existing specification in the United States includes requirements for shear connectors for encased steel sections. The provisions in this subsection require that shear connectors be provided to transfer all calculated axial forces between the structural steel and the concrete, neglecting the contribution of bond and friction. Friction between the structural steel and concrete is assumed to transfer the longitudinal shear stresses required to develop the plastic bending strength of the cross section. However, minimum shear studs should be provided according to the maximum spacing limit of 16 inches. Further information regarding the design of shear connectors for encased members is available (Furlong, 1997; Griffis, 1992a and 1992b).

3. The tie requirements in this section are essentially the same as those for composite columns in ACI 318 Chapter 10.

4. The requirements for longitudinal bars are essentially the same as those that apply to composite columns for low- and non-seismic design as specified in ACI 318. The distinction between load carrying and restraining bars is made to allow for longitudinal bars (restraining bars) that are provided solely for erection purposes and to improve confinement of the concrete. Due to interference with steel beams framing into the encased members, the restraining bars are often discontinuous at floor levels and, therefore, are not included in determining the column strength.

5. The requirements for the steel core are essentially the same as those for composite columns as specified in the LRFD Specification and ACI 318. In addition, earthquake damage to encased composite columns in Japan (Azizinamini and Ghosh, 1996) highlights the need to consider the effects of abrupt changes in stiffness and strength where encased composite columns transition into reinforced concrete columns and/or concrete foundations.

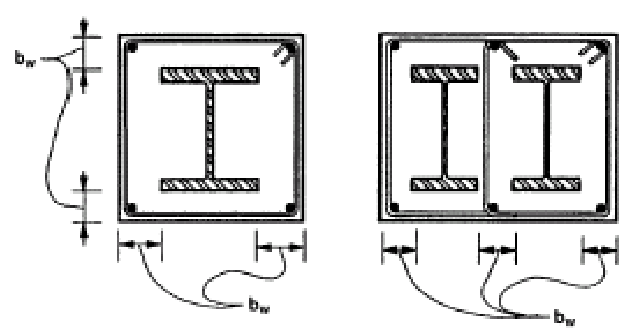


Fig. C-6.1. Effective widths for shear strength calculation of encased composite columns.

C6.4b. The more stringent tie spacing requirements for intermediate seismic systems follow those for reinforced concrete columns in regions of moderate seismicity as specified in ACI 318 Chapter 21 (Section 21.8). These requirements are applied to all composite columns for systems permitted in Seismic Design Category C to make the composite column details at least equivalent to the minimum level of detailing for columns in intermediate moment frames of reinforced concrete (FEMA, 1997a; ICC, 1997).

C6.4c. The additional requirements for encased composite columns used in special seismic systems are based upon comparable requirements for structural steel and reinforced concrete columns in systems permitted in Seismic Design Categories D and above (FEMA, 1997a; ICC, 1997). For additional explanation of these requirements, see the Commentaries for Part I in these Provisions and ACI 318 Chapter 21.

The minimum tie area requirement in Equation 6-2 is based upon a similar provision in ACI 318 Section 21.4.4, except that the required tie area is reduced to take into account the steel core. The tie area requirement in Equation 6-2 and related tie detailing provisions are waived if the steel core of the composite member can alone resist the expected (arbitrary point in time) gravity load on the column because additional confinement of the concrete is not necessary if the steel core can inhibit collapse after an extreme seismic event. The load combination of $1.0D + 0.5L$ is based upon a similar combination proposed as loading criteria for structural safety under fire conditions (Ellingwood and Corotis, 1991).

The requirements for composite columns in C-SMF are based upon similar requirements for steel and reinforced concrete columns in SMF (FEMA, 1997a; ICC, 1997). For additional commentaries, see Part I in these Provisions and ASCE 7.

The strong-column/weak-beam (SC/WB) concept follows that used for steel and reinforced concrete columns in SMF. Where the formation of a plastic hinge at the column base is likely or unavoidable, such as with a fixed base, the detailing should provide for adequate plastic rotational ductility. For Seismic Design Category E, special details, such as steel jacketing of the column base, should be considered to avoid spalling and crushing of the concrete.

Closed hoops are required to ensure that the concrete confinement and nominal shear strength are maintained under large inelastic deformations. The hoop detailing requirements are equivalent to those for reinforced concrete columns in SMF. The transverse reinforcement provisions are considered to be conservative since composite columns generally will perform better than comparable reinforced concrete columns with similar confinement. However, further research is required to determine to what degree the transverse reinforcement requirements can be reduced for composite columns. It should be recognized that the closed hoop and cross-tie requirements for C-SMF may require special details such as those suggested in Figure C-6.2 to facilitate the erection of the reinforcement around the steel core. Ties are required to be anchored into the confined core of the column to provide effective containment.

C6.5. Concrete-filled Composite Columns

The basic requirements and limitations for detailing and determining the design strength of filled composite columns are the same as those in LRFD Specification Chapter I. The limit of $A_s/A_g \geq 0.04$ is the same as that in the LRFD Specification and defines the limit of applicability of these Provisions. Although it is not intended in these Provisions that filled composite columns with smaller steel area ratios be prohibited, alternative provisions are not currently available.

C6.5a. The shear strength of the filled member is conservatively limited to the nominal shear yield strength of the steel tube because the actual shear strength contribution of the concrete fill has not yet been determined in testing. This approach is recommended until tests are conducted (Furlong, 1997; ECS, 1994). Even with this conservative approach, shear strength rarely governs the design of typical filled composite columns with cross-sectional dimensions up to 30 in. Alternatively, the shear strength for filled tubes can be determined in a manner that is similar to that for reinforced concrete columns with the steel tube considered as shear reinforcement and its shear yielding strength neglected. However, given the upper limit on shear strength as a function of concrete crushing in ACI 318, this approach would only be advantageous for columns with low ratios of structural steel to concrete areas (Furlong, 1997).

C6.5c. The more stringent slenderness criteria for the wall thickness in square or rectangular HSS is based upon comparable requirements from Part I in these Provisions for unfilled HSS used in SMF. Comparing the provisions in the LRFD Specification and Part I in these Provisions, the width/thickness ratio for unfilled HSS in SMF is about 80 percent of those for OMF. This same ratio of 0.8 was applied to the standard (non-seismic) b/t ratio for filled HSS in the LRFD Specification. The reduced slenderness criterion was imposed as a conservative measure until further research data becomes available on the cyclic response of filled square and rectangular tubes. More stringent D/t ratio limits for circular pipes are not applied as data are available to show the standard D/t ratio is sufficient for seismic design (Boyd et al., 1995; Schneider, 1998).

C7. COMPOSITE CONNECTIONS

C7.1 Scope

The use of composite connections often simplifies some of the special challenges associated with traditional steel and concrete construction. For example, compared to structural steel, composite connections often avoid or minimize the use of field welding, and compared to reinforced concrete, there are fewer instances where anchorage and development of primary beam reinforcement is a problem.

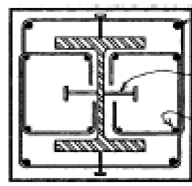


Fig. C-6.2. Example of a closed hoop detail for encased composite column.

Given the many alternative configurations of composite structures and connections, there are few standard details for connections in composite construction (Griffis, 1992b; Goel, 1992a; Goel, 1993). However, tests are available for several connection details that are suitable for seismic design. References are given in this Section of the Commentary and Commentary Sections C8 to C17. In most composite structures built to date, engineers have designed connections using basic mechanics, equilibrium, existing standards for steel and concrete construction, test data, and good judgment. The provisions in this Section are intended to help standardize and improve design practice by establishing basic behavioral assumptions for developing design models that satisfy equilibrium of internal forces in the connection for seismic design.

C7.2 General Requirements

The requirements for deformation capacity apply to both connections designed for gravity load only and connections that are part of the Seismic Force Resisting System. The ductility requirement for gravity load only connections is intended to avoid failure in gravity connections that may have rotational restraint but limited rotation capacity. For example, shown in Figure C-7.1 is a connection between a reinforced concrete wall and steel beam that is designed to resist gravity loads and is not considered to be part of the Seismic Force Resisting System. However, this connection is required to be designed to maintain its vertical shear strength under rotations and/or moments that are imposed by inelastic seismic deformations of the structure.

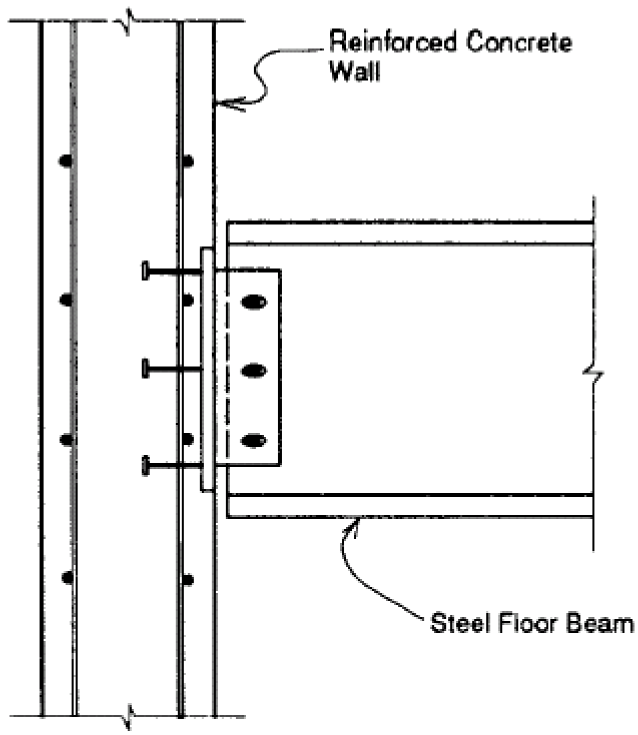


Fig. C-7.1. Steel beam-to-RC wall gravity load shear connection.

In calculating the required strength of connections based on the nominal strength of the connected members, allowance should be made for all components of the members that may increase the nominal strength above that usually calculated in design. For example, this may occur in beams where the negative moment strength provided by slab reinforcement is often neglected in design but will increase the moments applied through the beam-to-column connection. Another example is in concrete-filled tubular braces where the increased tensile and compressive strength of the brace due to concrete should be considered in determining the required connection strength. Because the evaluation of such conditions is case specific, these provisions do not specify any allowances to account for overstrength. However, as specified in Part I Section 6.2, calculations for the required strength of connections should, as a minimum, be made using the Expected Yield Strength of the connected steel member. Where connections resist forces imposed by yielding of steel in reinforced concrete members, ACI 318 Section 21.5 implies an expected yield strength equal to 1.25 F_y for reinforcing bars.

C7.3. Nominal Strength of Connections

C7.3.a. In general, forces between structural steel and concrete will be transferred by a combination of bond, adhesion, friction and direct bearing. Transfers by bond and adhesion are not permitted for nominal strength calculation purposes because: (1) these mechanisms are not effective in transferring load under inelastic load reversals; and (2) the effectiveness of the transfer is highly variable depending on the surface conditions of the steel and shrinkage and consolidation of the concrete.

Transfer by friction shall be calculated using the shear friction provisions in ACI 318 where the friction is provided by the clamping action of steel ties or studs or from compressive stresses under applied loads. Since the provisions for shear friction in ACI 318 are based largely on monotonic tests, the values are reduced by 25 percent where large inelastic stress reversals are expected. This reduction is a conservative requirement that does not appear in ACI 318 but is applied herein due the relative lack of experience with certain configurations of composite structures.

C7.3.b. In many composite connections, steel components are encased by concrete that will inhibit or fully prevent local buckling. For seismic deign where inelastic force reversals are likely, concrete encasement will be effective only if it is properly confined. One method of confinement is with reinforcing bars that are fully anchored into the confined core of the member (using requirements for hoops in ACI 318 Chapter 21). Adequate confinement also may occur without special reinforcement where the concrete cover is very thick. The effectiveness of the latter type of confinement should be substantiated by tests.

C7.3.c. For fully encased connections between steel (or composite) beams and reinforced concrete (or composite) columns such as shown in Figure C-7.2, the panel zone nominal shear strength can be calculated as the sum of contributions from the reinforced concrete and steel shear panels (see Figure C-7.3). This superposition of strengths for calculating the panel zone nominal shear strength is used in detailed design guidelines (Deierlein et al., 1989; ASCE, 1994) for composite connections that are supported by test data (Sheikh et al., 1989; Kanno and Deierlein, 1997; Nishiyama et al., 1990). Further information on the use and design of such connections is included in Commentary Section 9.

C7.3.d. Reinforcing bars in and around the joint region serve the dual functions of resisting calculated internal tension forces and providing confinement to the concrete. Internal tension forces can be calculated using established engineering models that satisfy equilibrium (e.g., classical beam-column theory, the truss analogy, strut and tie models). Tie requirements for confinement usually are based on empirical models of test data and past performance of structures (ACI, 1991; Kitayama et al., 1987).

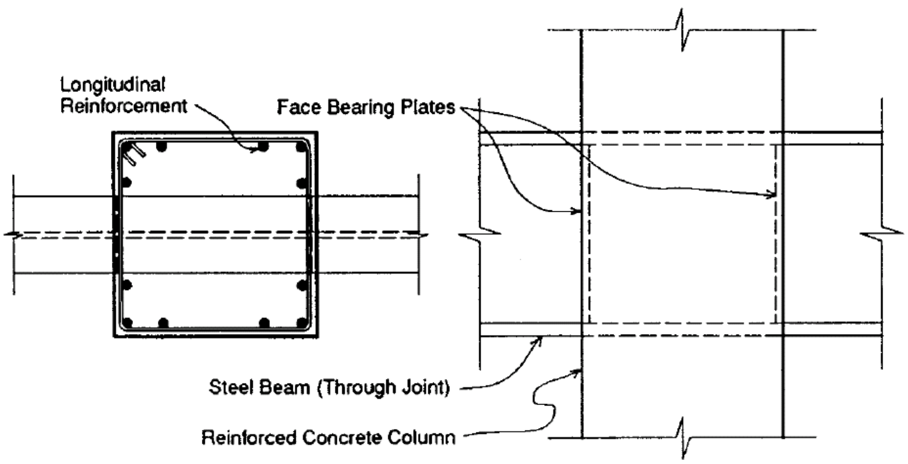


Fig. C-7.2. Reinforced concrete column-to-steel beam moment connection.

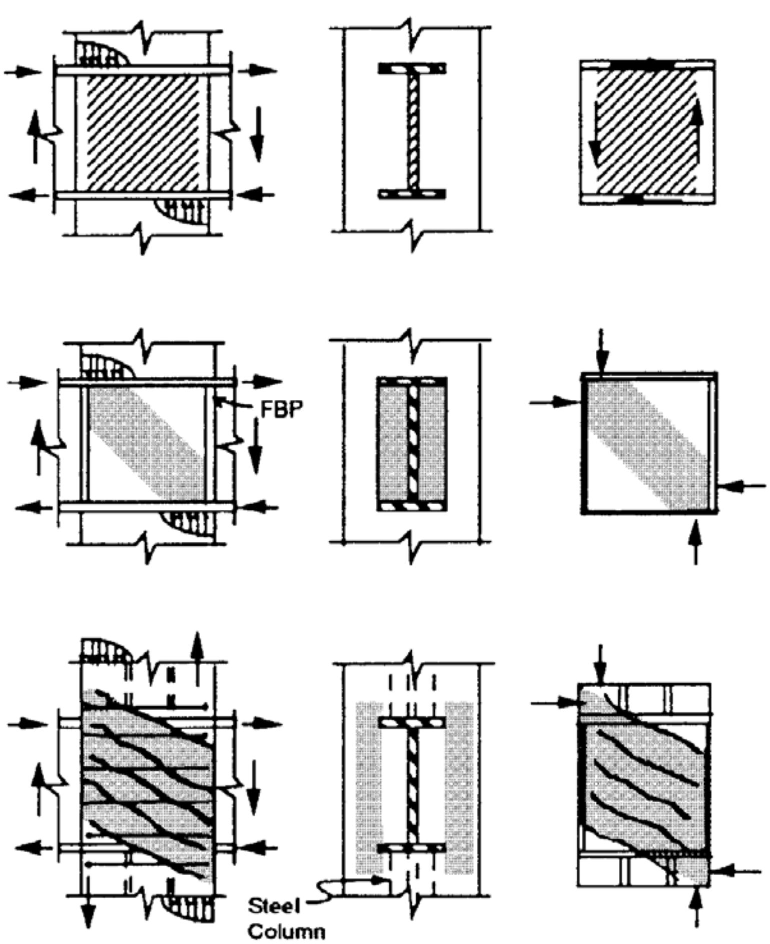


Fig. C-7.3. Panel shear mechanisms in steel beam-to-reinforced concrete column connections (Deierlein et al., 1989)

1. In connections such as those in C-PRMF, the force transfer between the concrete slab and the steel column requires careful detailing. For C-PRMF connections (see Figure C-7.4), the strength of the concrete bearing against the column flange should be checked. Only the solid portion of the slab (area above the ribs) should be counted, and the nominal bearing strength should be limited to $1.2f'_c$ (Ammerman and Leon, 1990). In addition, because the force transfer implies the formation of a large compressive strut between the slab bars and the column flange, adequate transverse steel reinforcement should be provided in the slab to form the tension tie. From equilibrium calculations, this amount should be the same as that provided as longitudinal reinforcement and should extend at least 12 in. beyond either side of the effective slab width.

2. Due to the limited size of joints and the congestion of reinforcement, it often is difficult to provide the reinforcing bar development lengths specified in ACI 318 for transverse column reinforcement in joints. Therefore, it is important to take into account the special requirements and recommendations for tie requirements as specified for reinforced concrete connections in ACI 318 Section 21.5 and in ACI (1991), Kitayama et al. (1987), Sheikh and Uzumeri (1980), Park et al. (1982) and Saatcioglu (1991). Test data (Sheikh et al., 1989; Kanno and Deierlein, 1997; Nishiyama et al., 1990) on composite beam-to-column connections similar to the one shown in Figure C-7.2 indicate that the face bearing (stiffener) plates attached to the steel beam provide effective concrete confinement.

3. As in reinforced concrete connections, large bond stress transfer of forces to column bars passing through beam-to-column connections can result in slippage of the bars under extreme loadings. Current practice for reinforced concrete connections is to control this slippage by limiting the maximum longitudinal bar sizes as described in ACI (1991).

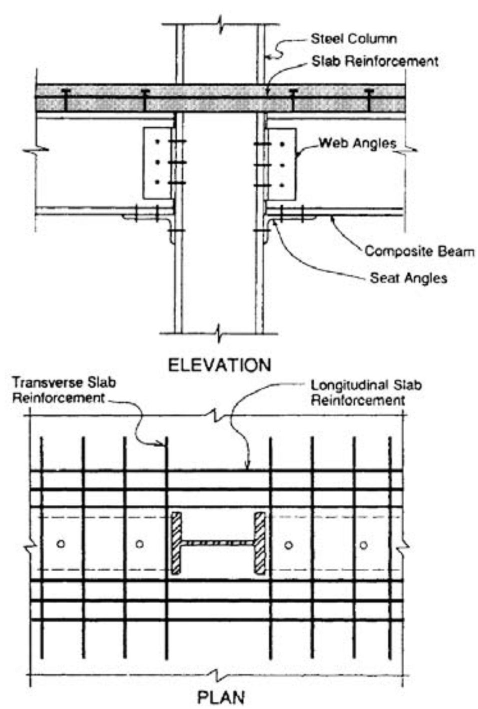


Fig. C-7.4. Composite partially restrained connection.

C8. COMPOSITE PARTIALLY RESTRAINED (PR) MOMENT FRAMES (C-PRMF)

Composite partially restrained (PR) frames consist of structural steel columns and composite steel beams that are interconnected with PR composite connections (Zandonini and Leon, 1992). PR composite connections utilize traditional steel frame shear and bottom flange connections and the additional strength and stiffness provided by the floor slab has been incorporated by adding shear studs to the beams and slab reinforcement in the negative moment regions adjacent to the columns (see Figure C-7.4). This results in a more favorable distribution of strength and stiffness between negative and positive moment regions of the beams and provides for redistribution of forces under inelastic action.

In the design of PR composite connections, it is assumed that bending and shear forces can be considered separately with the bending assigned to the steel in the slab and a bottom-flange steel angle or plate and the shear assigned to a web angle or plate. Design methodologies and standardized guidelines for C-PRMF frames and connections have been published (Ammerman and Leon, 1990; Leon and Forcier, 1992; Steager and Leon, 1993; Leon, 1990).

Subassemblage tests show that when properly detailed, the PR composite connections such as those shown in Figure C-7.4 can undergo large deformations without fracturing. The connections generally are designed with a yield strength that is less than that of the connected members to prevent local limit states, such as local buckling of the flange in compression, web crippling of the beam, panel zone yielding in the column and bolt or weld failures, from controlling. When these limit states are avoided, large connection ductilities should ensure excellent frame performance under large inelastic load reversals.

C-PRMF were originally proposed for areas of low to moderate seismicity in the eastern United States (Seismic Design Categories C and below). However, with appropriate detailing and analysis, C-PRMF can be used in areas of higher seismicity (Leon, 1990). Tests and analyses of these systems have demonstrated that the seismically induced forces on PR moment frames can be lower than those for FR moment frames due to: (1) lengthening in the natural period due to yielding in the connections and (2) stable hysteretic behavior of the connections (Nader and Astaneh, 1992; DiCorso, et al., 1989). Thus, in some cases, C-PRMF can be designed for lower seismic forces than OMF.

For frames up to four stories, the design should be made using an analysis that, as a minimum, accounts for the semi-rigid behavior of the connections by utilizing linear springs with reduced stiffness (Bjorhovde, 1984). The effective connection stiffness should be considered for determining member force distributions and deflections, calculating the building's period of vibration, and checking frame stability. Frame stability can be addressed using conventional effective buckling length procedures. However, the connection flexibility should be considered in determining the rotational restraint at the ends of the columns. For structures taller than four stories, drift and stability need to be carefully checked using analysis techniques that incorporate both geometric and connection non-linearities (Ammerman and Leon, 1990; Chen and Lui, 1991). PR composite connections can also be used as part of the gravity load system for braced frames provided that minimum design criteria such as those proposed by Leon and Ammerman (1990) are followed. In this case no height limitation applies, and the frame should be designed as a braced system.

Because the moments of inertia for composite beams in the negative and positive regions are different, the use of either value alone for the beam members in the analysis can lead to significant errors. Therefore, the use of a weighted average is recommended (Ammerman and Leon, 1990; Leon and Ammerman, 1990; Zaremba, 1988).

C9. COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

C9.1. Scope

Composite moment frames include a variety of configurations where steel or composite beams are combined with reinforced concrete or composite columns. In particular, composite frames with steel floor framing and composite or reinforced concrete columns have been used in recent years as a cost-effective alternative to frames with reinforced concrete floors (Furlong, 1997; Griffis, 1992b). For seismic design, composite moment frames are classified as either Special, Intermediate, or Ordinary depending upon the detailing requirements for the members and connections of the frame. As shown in Table II-C4-1, C-SMF are primarily intended for use in Seismic Design Categories D and above. Design and detailing provisions for C-SMF are comparable to those required for steel and reinforced concrete SMF and are intended to confine inelastic deformation to the beams. Since the inelastic behavior of C-SMF is comparable to that for steel or reinforced concrete SMF, the R and C_d values are the same as for those systems.

C9.3. Beams

The use of composite trusses as flexural members in C-SMF is not permitted unless substantiating evidence is provided to demonstrate adequate seismic resistance of the system. This limitation applies only to members that are part of the Seismic Force Resisting System and does not apply to joists and trusses that carry gravity loads only. Trusses and open web joists generally are regarded as ineffective as flexural members in lateral load systems unless either (1) the web members have been carefully detailed through a limit-state design approach to delay, control, or avoid overall buckling of compression members, local buckling, or failures at the connections (Itani and Goel, 1991) or (2) a strong-beam/weak-column mechanism is adopted and the truss and its connections proportioned accordingly (Camacho and Galambos, 1993). Both approaches can be used for one-story industrial-type structures where the gravity loads are small and ductility demands on the critical members can be sustained. Under these conditions and when properly proportioned, these systems have been shown to provide adequate ductility and energy dissipation capability.

C9.4 Moment Connections

A schematic connection drawing for composite moment frames with reinforced concrete columns is shown in Figure C-7.2 where the steel beam runs continuously through the column and is spliced away from the beam-to-column connection. Often, a small steel column that is interrupted by the beam is used for erection and is later encased in the reinforced concrete column (Griffis, 1992b). Since the late 1980s, over 60 large-scale tests of this type of connection have been conducted in the United States and Japan under both monotonic and cyclic loading (Sheikh et al., 1989; Kanno and Deierlein, 1997; Nishiyama et al., 1990). The results of these tests show that carefully detailed connections can perform as well as seismically designed steel or reinforced concrete connections. In particular, details such as the one shown in Figure C-7.2 avoid the need for field welding of the beam flange at the critical beam-to-column junction. Therefore, these joints are generally not susceptible to the fracture behavior that is now recognized as a critical aspect of welded steel moment connections. Tests have shown that, of the many possible ways of strengthening the joint, face bearing plates (see Figure C-7.2) attached to the beam are very effective for both mobilizing the joint shear strength of reinforced concrete and providing confinement to the concrete. Further information on design methods and equations for these composite connections is available in guidelines prepared by ASCE (Nishiyama et al., 1990). Note that while the scope of the current ASCE Guidelines (ASCE, 1994) limits their application to regions of low to moderate seismicity, recent test data indicate that the ASCE Guidelines are adequate for regions of high seismicity as well (Kanno and Deierlein, 1997; Nishiyama et al., 1990).

Connections between steel beams and encased composite columns (see Figure C-9.1) have been used and tested extensively in Japan where design provisions are included in Architectural Institute of Japan standards (AIJ, 1991). Alternatively, the connection strength can be conservatively calculated as the strength of the connection of the steel beam to the steel column. Or, depending upon the joint proportions and detail, where appropriate, the strength can be calculated using an adaptation of design models for connections between steel beams and reinforce concrete columns (ASCE, 1994). One disadvantage of this connection detail compared to the one shown in Figure C-7.2 is that, like standard steel construction, the detail in Figure C-9.1 requires welding of the beam flange to the steel column.

Connections to filled composite columns (see Figure C-9.2) have been used less frequently and only a few tests of this type have been reported (Azizinamini and Prakash, 1993). Where the steel beams run continuously through the composite column, the internal force transfer mechanisms and behavior of these connections are similar to those for connections to reinforced concrete columns (Figure C-7.2). Otherwise, where the beam is interrupted at the column face, special details are needed to transfer the column flange forces through the connection.

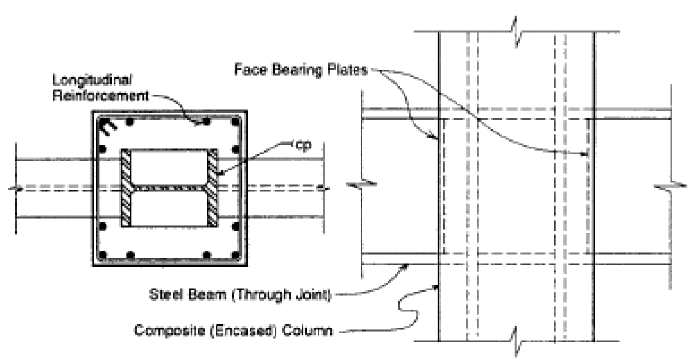


Fig. C-9.1. Steel band plates used for strengthening the joint.

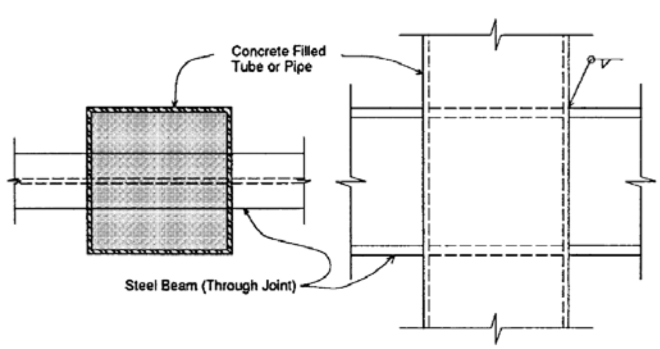


Fig. C-9.2. Composite (encased) column-to-steel beam moment connection.

These Provisions require that connections in C-SMF meet the same inelastic rotation capacity of 0.03 radians as required for steel SMF in Part I. In connection details where the beam runs continuously through the joint (Figure C-7.2) and the connection is not susceptible to fracture, then the connection design can be substantiated from available test data that is not subjected to requirements such as those described in Part I Appendix S. However, where the connection is interrupted and fracture is of concern, then connection performance should be substantiated following requirements similar to those in Part I Appendix S.

C10. COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)

The basic construction and connections for C-IMF are similar to C-SMF except that many of the seismic detailing requirements have been relaxed. C-IMF are limited for use in Seismic Design Category C and below, and provisions for C-IMF are comparable to those required for reinforced concrete IMF and between those for steel IMF and OMF. The R and C_d values for C-IMF are equal to those for reinforced concrete IMF and between those for steel IMF and OMF.

C11. COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)

C-OMF represent a type of composite moment frame that is designed and detailed following the LRFD Specification and ACI 318, excluding Chapter 21. C-OMF are limited to Seismic Design Categories A and B, and the design provisions are comparable to those for reinforced concrete and steel frames that are designed without any special seismic detailing. The R and C_d values for C-OMF are chosen accordingly.

C12. COMPOSITE ORDINARY BRACED FRAMES (C-OBF)

Composite braced frames consisting of steel, composite and/or reinforced concrete elements have been used in low- and high-rise buildings in regions of low and moderate seismicity. The C-OBF category is provided for systems without special seismic detailing that are used in Seismic Design Categories A and B. Because significant inelastic force redistribution is not relied upon in the design, there is no distinction between frames where braces frame concentrically or eccentrically into the beams and columns.

C13. COMPOSITE CONCENTRICALLY BRACED FRAMES (C-CBF)

C-CBF is one of the two types of composite braced frames that is specially detailed for Sesimic Design Categories C and above; the other is C-EBF (see Table II-C4-1). While experience using C-CBF is limited in high seismic regions, the design provisions for C-CBF are intended to result in behavior comparable to steel OCBF, wherein the braces often are the elements most susceptible to inelastic deformations (see Part I Commentary Section C14). The R and C_d values and usage limitations for C-CBF are the same as those for steel OCBF.

In cases where composite braces are used (either concrete filled or concrete encased), the concrete has the potential to stiffen the steel section and prevent or deter brace buckling while at the same time increasing the capability to dissipate energy. The filling of steel tubes with concrete has been shown to effectively stiffen the tube walls and inhibit local buckling (Goel and Lee, 1992). For concrete encased steel braces, the concrete should be sufficiently reinforced and confined to prevent the steel shape from buckling. It is recommended that composite braces be designed to meet all requirements of composite columns as specified in Sections 6.4a through 6.4c. Composite braces in tension should be designed based on the steel section alone unless test data justify higher strengths. Braces that are all steel should be designed to meet all requirements for steel braces in Part I of these Provisions. Reinforced concrete and composite columns in C-CBF are detailed with similar requirements to columns in C-SMF. With further research, it may be possible to relax these detailing requirements in the future.

Examples of connections used in C-CBF are shown in Figures C-13.1 through C-13.3. Careful design and detailing of the connections in a C-CBF is required to prevent failure before developing the strength of the braces in either tension or compression. All connection strengths should be capable of developing the full strength of the braces in tension and compression. Where the brace is composite, the added brace strength afforded by the concrete should be considered. In such cases, it would be unconservative to base the connection strength on the steel section alone. Connection design and detailing should recognize that buckling of the brace could cause excessive rotation at the brace ends and lead to local connection failure.

C14. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

Structural steel EBF have been extensively tested and utilized in seismic regions and are recognized as providing excellent resistance and energy absorption for seismic loads (see Part I Commentary Section C15). While there has been little use of C-EBF, the inelastic behavior of the critical steel Link should be essentially the same as for steel EBF and inelastic deformations in the composite or reinforced concrete columns should be minimal. Therefore, the R and C_d values and usage limitations for C-EBF are the same as those for steel EBF. As described below, careful design and detailing of the brace-to-column and Link-to-column connections is essential to the performance of the system.

The basic requirements for C-EBF are the same as those for steel EBF with additional provisions for the design of composite or reinforced concrete columns and the composite connections. While the inelastic deformations of the columns should be small, as a conservative measure, detailing for the reinforced concrete and encased composite columns are based upon those in ACI 318 Chapter 21. In addition, where Links are adjacent to the column, closely space hoop reinforcement is required similar to that used at hinge regions in reinforced concrete SMF. This requirement is in recognition of the large moments and force reversals imposed in the columns near the Links.

Satisfactory behavior of C-EBF is dependent on making the braces and columns strong enough to remain essentially elastic under forces generated by inelastic deformations of the Links. Since this requires an accurate calculation of the shear Link nominal strength, it is important that the shear Link region of the Link not be encased in concrete. Portions of the beam outside of the Link are permitted to be encased since an overstrength outside the Link would not reduce the effectiveness of the system. Shear Links are permitted to be composite with the floor or roof slab since the slab has a minimal effect on the nominal shear strength of the Link. The additional strength provided by composite action with the slab is important to consider, however, for long Links whose nominal strength is governed by flexural yielding at the ends of the Links (Ricles and Popov, 1989).

In C-EBF where the Link is not adjacent to the column, the concentric brace-to-column connections are similar to those shown for C-CBF (Figures C-13.1 through C-13.3). An example where the Link is adjacent to the column is shown in Figure C-14.1. In this case, the Link-to-column connection is similar to composite beam-to-column moment connections in C-SMF (see Section 9) and to steel coupling beam-to-wall connections (see Section 15).

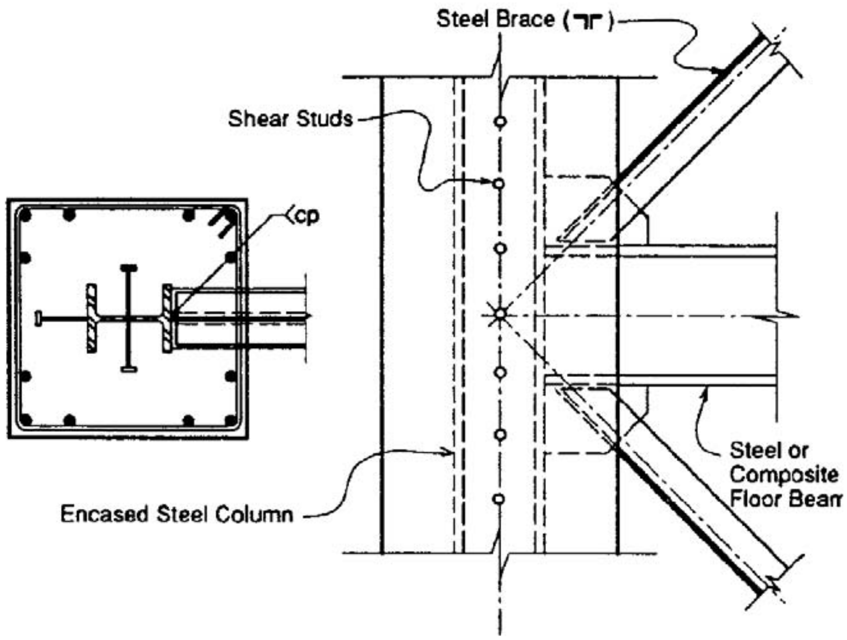


Fig. C-13.1. Reinforced concrete (or composite) column-to-steel concentric brace.

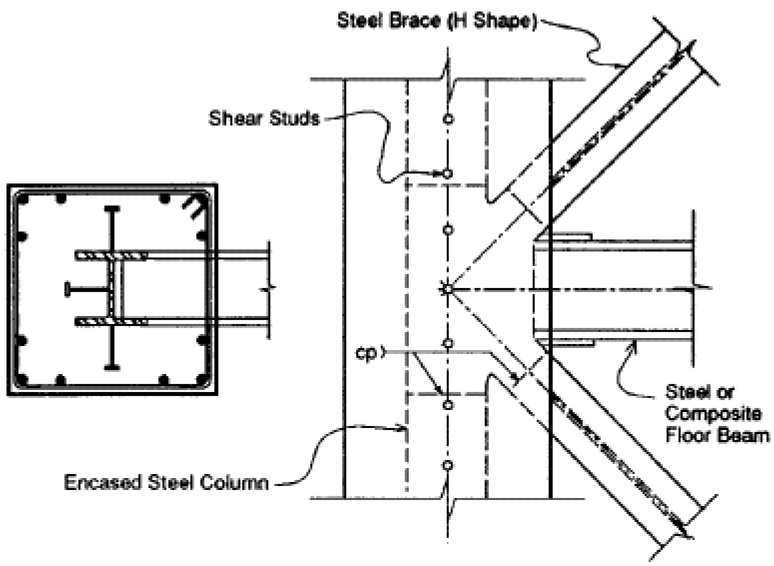


Fig. C-13.2. Reinforced concrete (or composite) column-to-steel concentric brace.

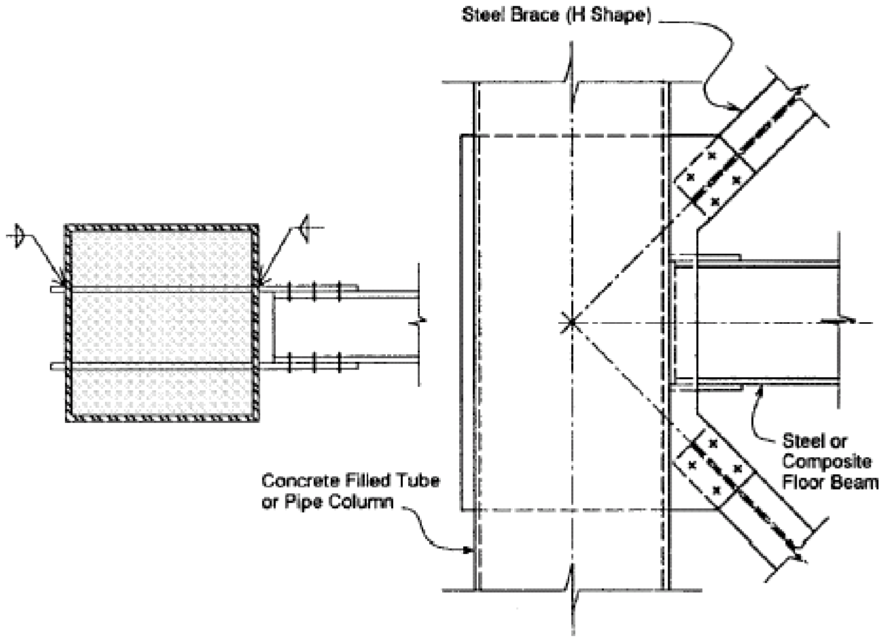


Fig. C-13.3. Concrete filled tube or pipe column-to-steel concentric base.

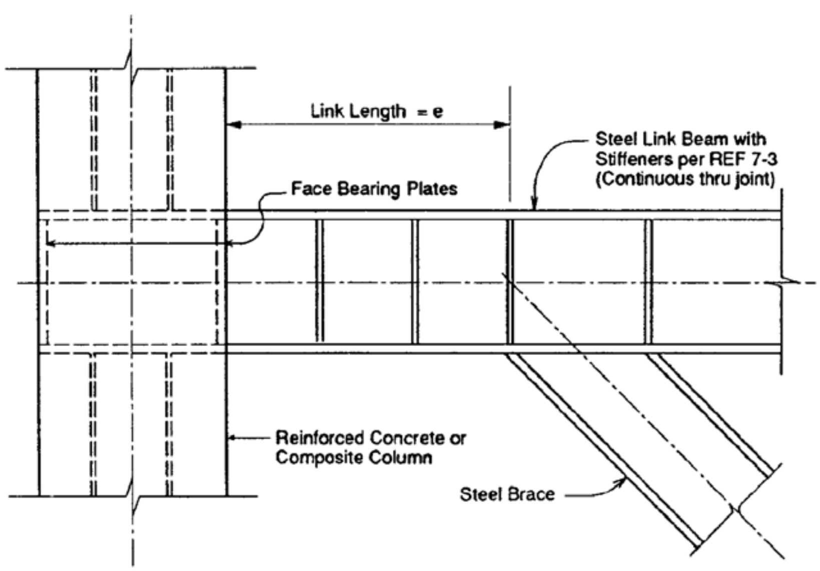


Fig. C-14.1. Reinforced concrete (or composite) column-to-steel eccentric brace.

C15. ORDINARY REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-ORCW)

The provisions in this Section apply to three variations of structural systems using reinforced concrete walls. One type is where reinforced concrete walls serve as infill panels in what are otherwise steel or composite frames. Examples of typical sections at the wall-to-column interface for such cases are shown in Figures C-15.1 and C-15.2. The details in Figure C-15.2 also can occur in the second type of system where encased steel sections are used as vertical reinforcement in what are otherwise reinforced concrete shear walls. Finally, the third variation is where steel or composite beams are used to couple two or more reinforced concrete walls. Examples of coupling beam-to-wall connections are shown in Figures C-15.3 and C-15.4. When properly designed, each of these systems should have shear strength and stiffness comparable to those of pure reinforced concrete shear wall systems. The structural steel sections in the boundary members will, however, increase the in-plane flexural strength of the columns and delay flexural hinging in tall walls. R and C_d values for reinforced concrete shear walls with composite elements are the same as those for traditional reinforced concrete shear wall systems. Requirements in this section are for ordinary reinforced concrete shear walls that are limited to use in Seismic Design Categories C and below; requirements for special reinforced concrete shear walls permitted in Seismic Design Categories D and above are given in Section 16.

For cases where the reinforced concrete walls frame into non-encased steel shapes (Figure C-15.1), mechanical connectors are required to transfer vertical shear between the wall and column, and to anchor the wall reinforcement. Additionally, if the wall elements are interrupted by steel beams at floor levels, shear connectors are needed at the wall-to-beam interface. Tests on concrete infill walls have shown that if shear connectors are not present, story shear forces are carried primarily through diagonal compression struts in the wall panel (Chrysostomou, 1991). This behavior often includes high forces in localized areas of the walls, beams, columns, and connections. The shear stud requirements will improve performance by providing a more uniform transfer of forces between the infill panels and the boundary members.

Two examples of connections between steel coupling beams to concrete walls are shown in Figures C-15.3 and C-15.4. The requirements for coupling beams and their connections are based largely on recent tests of unencased steel coupling beams (Harries, et al., 1993; Shahrooz et al., 1993). These test data and analyses show that properly detailed coupling beams can be designed to yield at the face of the concrete wall and provide stable hysteretic behavior under reversed cyclic loads. Under high seismic loads, the coupling beams are likely to undergo large inelastic deformations through either flexural and/or shear yielding. However, for the ordinary class of shear wall, there are no special requirements to limit the slenderness of coupling beams beyond those in the LRFD Specification. More stringent provisions are required for the special class of shear wall (see Section 16).

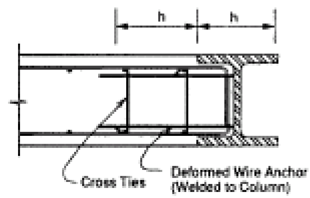


Fig. C-15.1. Partially encased steel boundary element.

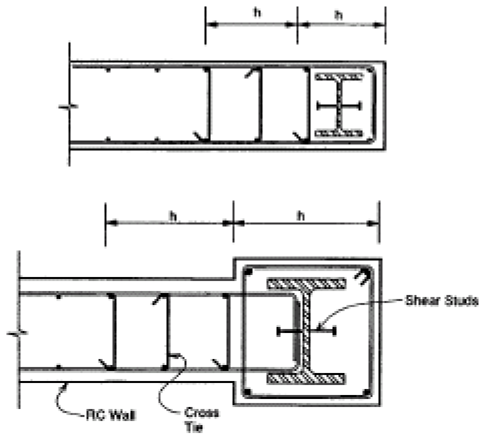


Fig. C-15.2. Fully encased composite boundary element.

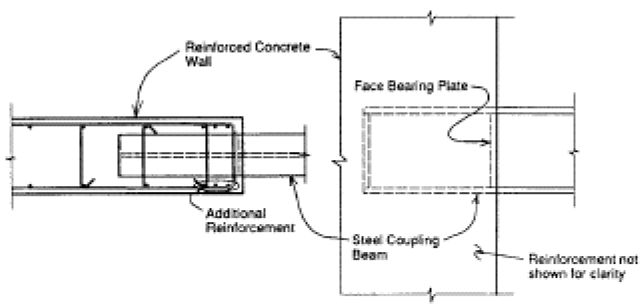


Fig. C-15.3. Steel coupling beam to reinforced concrete wall.

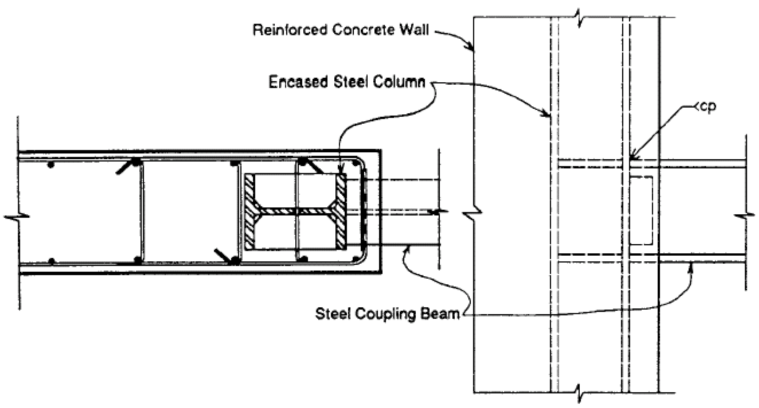


Fig. C-15.4. Steel coupling beam to reinforced concrete wall with composite boundary member.

C16. SPECIAL REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS (C-SRCW)

Additional requirements are given in this section for composite features of reinforced concrete walls classified as special that are permitted in Seismic Design Categories D and above. These provisions are applied in addition to those explained in the commentary to Section 15. As shown in Table C4.1, the *R* value for special reinforced concrete walls is larger than for ordinary walls.

Concerns have been raised that walls with encased steel boundary members may have a tendency to split along vertical planes inside the wall near the column. Therefore, the provisions require that transverse steel be continued into the wall for the distance $2h$ as shown in Figures C-15.1 and C-15.2.

As a conservative measure until further research data are available, strengths for shear studs to transfer force into the structural steel boundary members are reduced by 25 percent from their Static Yield Strength. This is done because provisions in the Specification and most other sources for calculating the nominal strength of shear studs are based on static monotonic tests. The 25 percent reduction in stud strengths need not apply to cases where the steel member is fully encased since the provisions conservatively neglect the contribution of bond and friction between the steel and concrete.

Several of the requirements for Links in steel EBF are applied to coupling beams to insure more stable yielding behavior under extreme earthquake loading. It should be noted, however, that the Link requirements for steel EBF are intended for unencased steel members. For encased coupling beams, it may be possible to reduce the web stiffener requirements of Section 16.3.a, which are the same as those in Part I Section 15.3a, but currently, there are no data available that provides design guidance on this.

C17. COMPOSITE STEEL PLATE SHEAR WALLS (C-SPW)

Steel plate reinforced composite shear walls can be used most effectively where story shear forces are large and the required thickness of conventionally reinforced shear walls is excessive. The provisions limit the shear strength of the wall to the yield strength of the plate because there is insufficient basis from which to develop design rules for combining the yield strength of the steel plate and the reinforced concrete panel. Moreover, since the shear strength of the steel plate usually is much greater than that of the reinforced concrete encasement, neglecting the contribution of the concrete does not have a significant practical impact. The NEHRP Provisions assign structures with composite walls a slightly higher *R*-value than special reinforced concrete walls because the shear yielding mechanism of the steel plate will result in more stable hysteretic loops than for reinforced concrete walls (see Table II-C4-1). The *R*-value for C-SPW is also the same as that for light frame walls with shear panels.

Two examples of connections between composite walls to either steel or composite boundary elements are shown in Figures C-17.1, C-17.2, and C-17.3. The provisions require that the connections between the plate and the boundary members (columns and beams) be designed to develop the full yield strength of the plate. Minimum reinforcement in the concrete cover is required to maintain the integrity of the wall under reversed cyclic loading and out-of-plane forces. Until further research data are available, the minimum required wall reinforcement is based upon the specified minimum value for reinforced concrete walls in ACI 318.

The thickness of the concrete encasement and the spacing of shear stud connectors should be calculated to ensure that the plate can reach yield prior to overall or local buckling. It is recommended that overall buckling of the composite panel be checked using elastic buckling theory using a transformed section stiffness of the wall. For plates with concrete on only one side, stud spacing requirements that will meet local plate buckling criteria can be calculated based upon *h/t* provisions for the shear design of webs in steel girders. For example, in LRFD Specification Section F2.2, the limiting *h/t* value specified for compact webs subjected to shear is $h/t = 187\sqrt{k/F_y}$. Assuming a conservative value of the plate buckling coefficient $k = 5$ and F_y =50 ksi, this equation gives the limiting value of $h/t \leq 59$. For a 3/8-in.-thick plate, this gives a maximum value of $h = 22$ in. that is representative of the maximum center-to-center stud spacing that should suffice for the plate to reach its full shear yielding strength.

Careful consideration should be given to the shear and flexural strength of wall piers and of spandrels adjacent to openings. In particular, composite walls with large door openings may require structural steel boundary members attached to the steel plate around the openings.

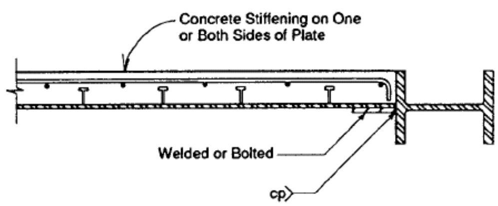


Fig. C-17.1. Concrete stiffened steel shear wall with steel boundary member.

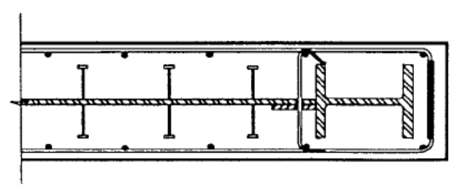


Fig. C-17.2. Concrete stiffened steel shear wall with composite (encased) boundary member.

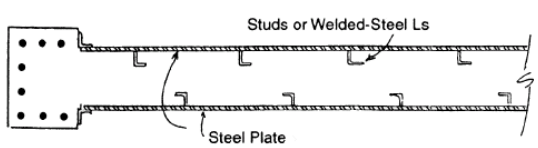


Fig. C-17.3. Concrete filled composite shear wall with two steel plates.

Part III - Allowable Stress Design (ASD) Alternative

C1. SCOPE

Part III has been included in these Seismic Provisions for designers that choose to use ASD in the seismic design of steel structures. As noted in Part I, the seismic requirements are collateral provisions related to the LRFD Specification. Part I is based upon the limit-state seismic load model used in the 1997 NEHRP Provisions. Since the seismic requirements in Part I are based upon the expected nonlinear performance of a structure, the use of ASD in its traditional form is somewhat complicated because a knowledge of design strengths, not allowable stresses, is required to assure that connectors have sufficient strength to allow nonlinear behavior of the connected member(s).

The provisions in Part III allow for the selection of members in an ASD format that still provides for the performance intended in Part I. Part III is intended as an overlay to Part I and, when using ASD, the designer will use Part I for the seismic design of a structure except where a section is replaced by or modified by a section shown in Part III.

Provisions have not been included for the use of ASD with the composite structural steel and reinforced concrete systems, members and connections in Part II because ACI 318 is in limit-states format.

C4. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

C4.1. Loads and Load Combinations

As this specification is being prepared, there continues to be differences in several key codes and standards on the appropriate load factor to be applied to E when using allowable stress design. A limit-state based seismic load model was introduced into ASCE 7 for the first time in the 1993 edition that was based upon the 1991 NEHRP *Recommended Provisions for Seismic Regulations for New Buildings.* ASCE 7-88 and its predecessor documents used a working-load seismic load model and a corresponding load factor on E of 1.5 for LRFD and 1.0 for ASD. In ASCE 7-93, the seismic load model was changed to a limit state basis and the load factor on E was set at 1.0 for both ASD and LRFD as documented in the commentary therein. At the same time, the load model in the Uniform Building Code continued to be ASD based and was not changed to a limit state model until the publication of the 1997 UBC. There, the load factor on E was set at 1.0 for LRFD and $E/1.4$ for ASD. It is expected that with the rapidly changing code environment some of this confusion will begin to be resolved with the development of the 2000 International Building Code.

As mentioned above, load factors on E are inconsistent throughout the codes and standards in the U.S. and the designer needs to be aware of using the appropriate load factor for E. However, where the code or standard contains a load factor on E that differs from those in Load Combinations 4-1 and 4-2, the designer is encouraged to use a load factor consistent with the governing code or standard.

C4.2. Nominal Strengths

The procedures in this section provide a methodology for the conversion of allowable stresses into nominal strengths, in most cases by removing the factor of safety from the ASD equations. When doing so, use of the 1/3 increase from ASD Specification Section A5.2 is not permitted. These nominal strengths are converted to design strengths when multiplied by the resistance factors given in Part III Section 4.3. In general, the resistance factors given are consistent with those in the LRFD Specification.

The remainder of the provisions in Part III translate the provisions of Part I into ASD terminology and correlate with the appropriate sections of ASD.

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